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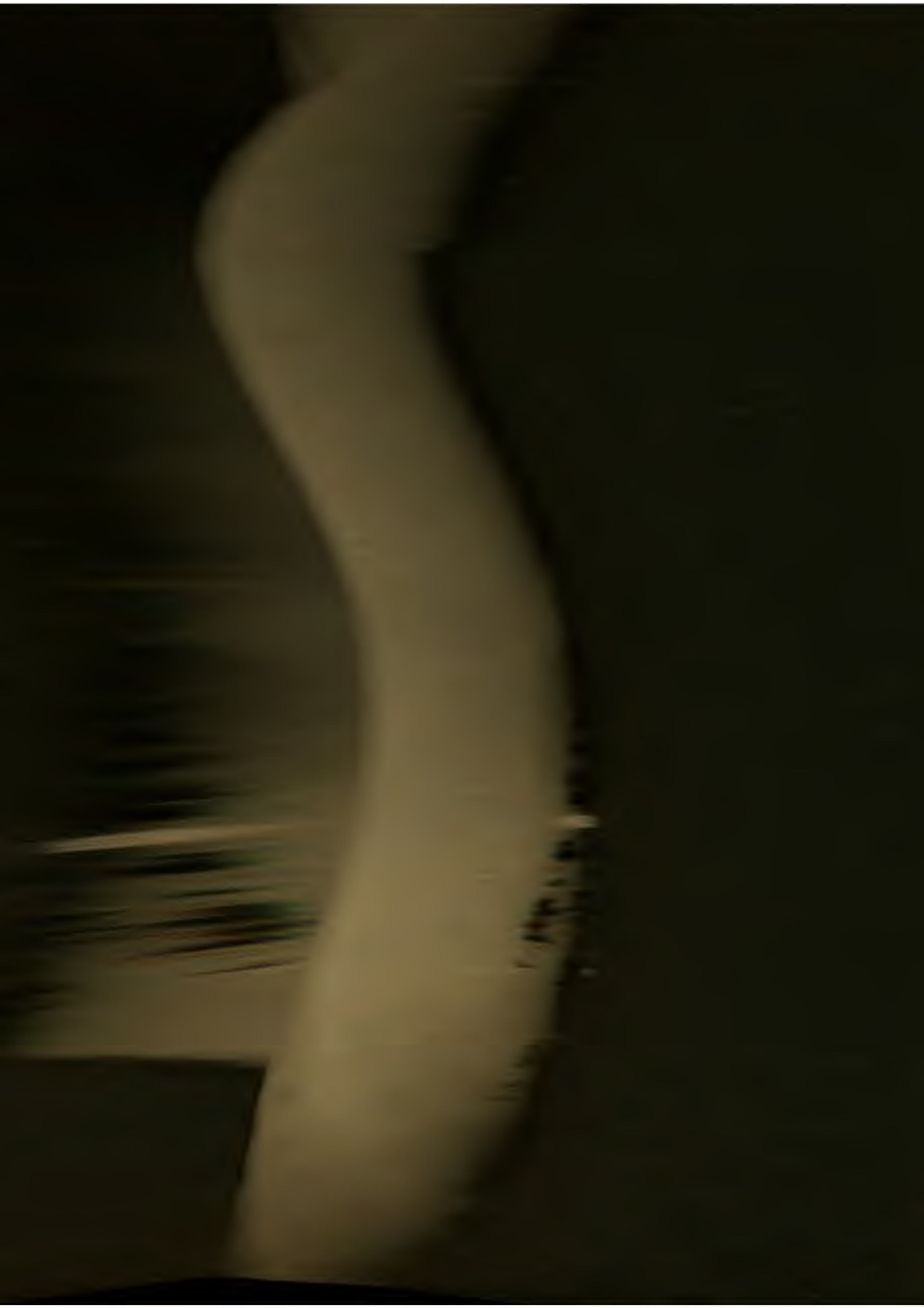

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THE DESIGN OF WALLS, BINS

AND

GRAIN ELEVATORS

BY

MILO S. KETCHUM, C.E.

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PREFACE.

The aim in writing this book has been to present a systematic analysis of the stresses due to granular materials together with a discussion of the principles of design and the details of structures which contain the granular materials. Since a thorough knowledge of the theory of the retaining wall is essential to a correct understanding of the theory of pressures in bins, the design of retaining walls is taken up first. The design of bins for coal, ore, etc., is then taken up in detail, and is followed by the design of grain bins and elevators. The sequence of the different subjects is the same as the order in which the work was developed by the author, and it is thought to be logical and consistent. While algebraic methods for calculating stresses have been given due consideration, attention is called to the graphic methods for calculating the pressures on retaining walls, and in hopper bins.

Reinforced concrete is now very extensively used in the construction of retaining walls, bins, and grain elevators, and it has therefore appeared advisable to give a discussion of the theory of reinforced concrete and to develop the formulas necessary for use in design.

Experiments on the pressure of grain in bins have given pressures that agree closely with the values obtained by calculation for grain at rest, and that show the influence of grain in motion. Very few experiments have been made, however, on granular materials other than grain with sufficient care to give anything like a reliable check on calculated pressures. All experiments on the pressure of granular materials available to the author have been included in the discussion. The subject of costs has been given considerable attention and numerous examples of actual structures are described in detail.

In the author's practice and teaching he has felt the need of a systematic treatise on bin design, and this book has been written as a first contribution to the subject.

The author is under obligations to many sources to which proper credit has been given in the text of the book. To obtain the latest practice in bin and grain elevator design the author has had to depend very largely upon engineers that make a specialty of this line of work. The author is under obligations to many engineers for data and plans; he especially wishes to thank Mr. H. A. Fitch, Chief Engineer the Minneapolis Steel & Machinery Co.; Mr. John S. Metcalf, President, The John S. Metcalf Co.; Mr. James MacDonald, President, The MacDonald Engineering Co.; and Mr. A. F. Robinson, Bridge Engineer Santa Fe Ry. Credit is due Mr. Howard C. Ford, Instructor in Civil Engineering in the University of Colorado, for assistance in preparing the drawings and in reading proof.

M. S. K.

January 26, 1907,
UNIVERSITY OF COLORADO,
BOULDER, COLO.

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THE DESIGN OF WALLS, BINS AND GRAIN ELEVATORS.

INTRODUCTION.

Probably no subject with which the civil engineer has had to deal has evoked so much discussion as the design of retaining walls. One class of writers has evolved elaborate mathematical theories, on the one hand, while another class has ridiculed all mathematical treatment, and has used "rule of thumb" methods for design. Many of the mathematical enthusiasts have failed to appreciate actual conditions of the wall and the filling; while most of the "rule of thumb" writers show an entire lack of knowledge of the fundamental theories underlying a theoretical discussion of the pressure on retaining walls.

While an exact determination of the pressure on retaining walls is difficult if not impossible on account of the lack of definite data on the condition of the filling and the foundation, it does not follow that a mathematical solution is useless. Experiments on homogeneous granular materials like wheat, sand, etc., have shown that the pressure follows laws which are definite and exact.

An argument against retaining wall formulas has been that many walls which theory shows are on the point of failure, have not failed, but would seem to have an ample factor of safety. This argument is threadbare, for what bridge engineer has not cudgeled his brain to find why certain bridges have not fallen down; and who is there that would presume to say that the application of theoretical analysis to bridge design is of no value. The answer to both examples is the same, the fault is more in the data than in the theory—we have failed to use brains in the analysis.

While the theory of Rankine as given in the following discussion is by no means perfect, it is the author's opinion that its intelligent use will put the design of retaining walls on as scientific a basis as the design of most engineering structures. Bridges are designed for the heaviest load that they are expected to carry, and not for the lightest load. Retaining wall formulas give maximum pressures for the conditions, which are certainly the ones upon which to base the design.

When retaining walls were built of cheap masonry, "rule of thumb" methods might do; but with the introduction of reinforced concrete retaining walls, and the storage of granular materials in bins, a mathematical treatment is certainly necessary. With improved methods of handling grain and other granular materials it has become necessary to design bins on economical lines. While the problem of bin design differs from the design of retaining walls in many ways, a thorough knowledge of the theory of the retaining wall is necessary to a correct understanding of the problem.

In the following treatise the author has endeavored to place the design of walls and bins on a scientific basis. The discussion will be given in three parts: Part I, The Design of Retaining Walls; Part II, The Design of Coal Bins, Ore Bins, etc.; Part III, The Design of Grain Bins and Elevators.

PART I.

THE DESIGN OF RETAINING WALLS.

INTRODUCTION.

A retaining wall is a structure which sustains the lateral pressure of earth or some other granular mass which possesses some frictional stability. The pressure of the material supported will depend upon the material, the manner of depositing in place, and upon the amount of moisture, and will vary from zero to the full hydraulic pressure. If dry clay is loosely deposited behind the wall it will exert full pressure due to this condition. In time the earth becomes consolidated and cohesion and moisture make a solid clay, which often causes the bank to shrink away from the wall and there will be no pressure exerted. On the other hand, if a considerable amount of water is applied, the normal pressure rapidly increases and approaches that due to a liquid having the same specific gravity.

In this discussion the first condition only will be considered and the earth or other filling will be assumed to be a dry, granular mass, a semi-fluid, without cohesion, and to be held in place by the friction of the particles on each other.

To fully determine the pressure of the filling on the retaining wall it is necessary that the resultant of the pressure be known (a) in amount, (b) in direction, and (c) in point of application. Many theories have been proposed for finding the pressure, each differing somewhat in the assumptions as to the direction and point of application of the resultant pressure. The most important theories are as follows:

(1) **Rankine's Theory.**—In this theory the filling is assumed to consist of an incompressible, homogeneous, granular mass, without cohesion, the particles are held in position by friction on each other;

the mass being of indefinite extent, having a plane top surface, and resting on a homogeneous foundation, and being subjected to its own weight. These assumptions lead to the ellipse of stress and make the resultant pressure on a vertical wall parallel to the top surface. The pressure on other than vertical walls can be determined by the ellipse of stress.

(2) **Weyrauch's Theory.**—In this theory the filling is assumed to be without cohesion and to be held in equilibrium by friction of the particles on each other. It is also assumed that the forces upon any imaginary plane-section through the mass of earth have the same direction. These assumptions lead to two formulas, one giving the amount of the thrust, and the other giving its direction, the angle that the resultant makes with a normal to the wall. The formulas deduced by Weyrauch may be obtained more simply by means of the ellipse of stress, and are therefore subject to the same limitations.

(3) **Coulomb's Theory.**—In this theory it is assumed that there is a wedge having the wall as one side and a plane called the plane of rupture as the other side, which exerts a maximum thrust on the wall. The plane of rupture lies between the angle of repose of the filling and the back of the wall. It may coincide with the plane of repose. For a wall without surcharge (horizontal surface back of the wall) and a vertical wall, the plane of rupture bisects the angle between the plane of repose and the back of the wall. This theory does not determine the direction of the thrust, and leads to many other theories having assumed directions for the resultant pressure.

(4) **Cain's Theory.**—Professor William Cain assumes that the resultant thrust makes an angle with the normal equal to ϕ', the angle of friction of the filling on the back of the wall, or equal to ϕ, the angle of repose of the filling, if ϕ' is greater than ϕ.

Other authorities assume that the resultant thrust is normal to the back of the wall. For a smooth vertical wall without surcharge, all of the above formulas lead to the same result for the amount, direction, and point of application of the resultant thrust.

(5) **Trautwine's Theory.**—In Trautwine's Engineer's Pocket Book it is assumed, for a wall nearly vertical, that the plane of rupture in all cases bisects the angle between the plane of repose and the back of the wall. This theory gives correct results for a vertical wall with horizontal surface back of the wall, but is in error for all other cases.

In the following discussion the formulas for the thrust of the filling on retaining walls will be deduced (a) according to Rankine's Theory, and (b) a general formula will be derived based on Coulomb's Theory of a Maximum Wedge.

Nomenclature.—The following nomenclature will be used:

ϕ = the angle of repose of the filling.
ϕ' = the angle of friction of the filling on the back of the wall.
θ = the angle between the back of the wall and a horizontal line passing through the heel of the wall and extending from the back into the fill.
δ = angle of surcharge, the angle between the surface of the filling and the horizontal; δ is positive when measured above and negative when measured below the horizontal.
z = the angle which the resultant earth-pressure makes with a normal to the back of the wall.
h = the vertical height of the wall in feet.
d = the width of the base of the wall in feet.
b = the distance from the center of the base to the point where the resultant pressure, E, cuts the base.
P = the resultant earth-pressure per foot of length of wall.
E = the resultant of the earth-pressure and the weight of the wall.
w = the weight of the filling per cubic foot.
W = the total weight of the wall per foot of length of wall.
p_1 = the pressure on the foundation due to direct pressure.
p_2 = the pressure on the foundation due to bending moments.
p = the resultant pressure on the foundation due to direct and bending forces.
y = the depth of foundation below the earth surface.

CHAPTER I.

RANKINE'S THEORY.

Introduction.—In this theory the filling is assumed to consist of an incompressible, homogeneous, granular mass, without cohesion, the particles are held in position by friction on each other; the mass being of indefinite extent, having a plane top surface, and resting on a homogeneous foundation, and being subjected to its own weight. These assumptions lead to the ellipse of stress and make the resultant pressure on a vertical wall parallel to the top surface. The pressure on other than vertical walls can be determined by the ellipse of stress.

ELLIPSE OF STRESS.—It will now be necessary to investigate the relations between the stresses in an unconfined, incompressible, granular mass, which is held together by friction of the particles on each other, and which has no cohesion.

Stress.—If a body be conceived to be divided into two parts by a plane traversing it in any direction, the force exerted between these two parts at the plane of division is *an internal stress.* Stress is force distributed over an area in such a way as to be in equilibrium. A state of internal stress is or may be exerted upon every plane passing through a point at which such a state exists.

Plane Stress.—A plane stress is a stress that is parallel to a given plane; for example, let the plane of the paper be this plane and let the stress acting on every plane which is at right angles to the plane of the paper be parallel to the plane of the paper, then such a stress is a plane stress. Plane stress only will be considered in this discussion. The obliquity of a stress is the angle between the direction of the stress and a normal to the plane on which the stress acts.

State of Stress.—The state of plane stress at any point in a body is completely defined when the intensity and obliquity of the stress on any two given planes passing through that point are known. Stress can be resolved into components the same as force. An oblique stress can be resolved into a normal component (tension or compression) and a component along the plane called the tangential component, or shear.

Conjugate Stresses.—If any state of stress exists at a point in a body the stress acting on a plane through the point will be parallel to a second plane through the point, and the stress acting on the second plane will be parallel to the first plane. Stresses so related are called *conjugate stresses.*

In Fig. 1 let the elementary prism be in equilibrium when acted upon by the stress p parallel to A-C, and stress q direction unknown. It is required to find the direction of stress q. Now the stress p acting on the faces A-B and C-D must be a pair of forces in equlibrium; then to have equilibrium stress q acting on the faces A-C and A-D must be a pair of forces in equilibrium, which can only occur when stress q is parallel to A-B and C-D. Conjugate stresses may be like (both tension or both compression) or unlike (one tension and the other compression).

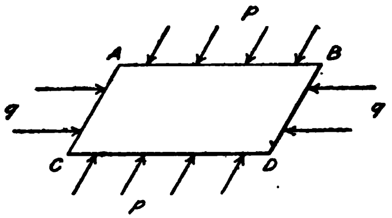


FIG. 1.

Principal Stresses.—In any state of stress there is one pair of conjugate stresses at right angles to each other; *i. e.,* there are two planes at right angles to each other on which the stress is normal only. Stresses so related are *principal stresses.* For consider a plane at the point to rotate, and there will be some position where the stress on the plane is normal. The plane at right angles will have a conjugate stress which is also normal. It will be shown that one of the principal stresses is the greatest stress and the other principal stress is the least stress at the point. Any possible state of stress can be completely defined by principal stresses.

Tangential Components.—In Fig. 2 let p and q be tangential components acting on the right prism A-B-C-D and producing equilibrium. Then for equilibrium p must equal q. For the stress on A-B $= AB.q$, and the moment of the couple $AB.q$ is $AB.q.AC$. Also the stress on A-$C = AC.p$, and the moment of the couple $AC.p$ is $AC.p.AB$. But for equilibrium the moment of the two couples must be equal, which can only be if $p = q$.

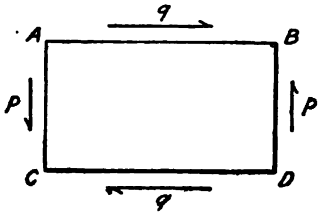


FIG. 2.

Case I.—*Equal-like Principal Stresses.*—If a pair of principal stresses at a point be like (both compression or both tension), and be equal in intensity, the stress on a third plane through the point is of the same intensity and is normal to the plane.

In (a) Fig. 3, p and q are equal compressive stresses acting on the principal planes A-A and B-B, respectively, through the point O. It is required to find the intensity and direction of the stress, r, acting on the plane C-C through the point O.

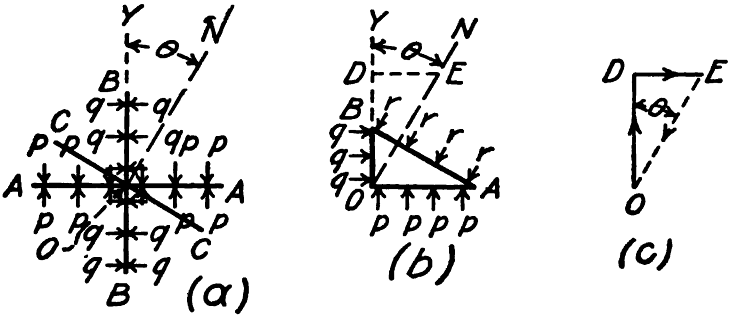


FIG. 3.

Let AOB in (b), Fig. 3, be a differential triangular prism at O, having its faces in the planes. The prism is then in equilibrium under the action of the forces p, q, and r, acting on the faces OA, OB, and AB, respectively. In (c) lay off $OD =$ total stress on $AO = p.OA$, and lay off $DE =$ total stress on $OB = q.OB$. Now complete the force triangle ODE, and $OE =$ the equilibrant $= AB.r$, in amount and direction.

$$\text{Now } \tan DOE = \frac{q.OB}{p.OA} = \frac{OB}{OA}, \text{ since } p = q.$$

Therefore, angle $DOE =$ angle $OAB = \theta$, and OE is perpendicular to AB.

$$\begin{aligned} \text{Also } OE^2 &= DO^2 + DE^2, \\ &= p.^2OA^2 + q.^2OB^2, \\ &= p^2(OA^2 + OB^2), \\ &= p.^2AB^2, \\ \text{and } OE &= p.AB \end{aligned}$$

But $OE = r.AB$, and therefore $r = p = q$, which proves the theorem. The stresses in Case 1 are *fluid stresses.*

Case 2.—Equal-unlike Principal Stresses.—If a pair of principal stresses at a point be unlike (one compression and the other tension), and be of equal intensity, the resultant on any plane through the point is of the same intensity, and is inclined to the normal to the plane of principal stress at an angle θ, but on the opposite side from the resultant in Case 1.

In Fig. 4, (a) shows the planes and forces acting through the point O. Let OAB in (b) be a differential triangular prism at O with its faces in the given planes. The prism is in equilibrium under the action of the forces p, q, and r, acting on the faces OA, OB, and AB, respectively. In (c) lay off $OD = AO.p$, and $ED = OB.q$, and OE will be the equilibrant acting on the face $AB = AB.r$.

$$\text{Now } \tan EOD = \frac{ED}{OD} = \frac{q.OB}{p.OA} = \frac{OB}{OA}, \text{ since } p = q.$$

Therefore, angle $EOD = \theta$, and EO makes an angle with the normal to OA equal to θ, but on the opposite side from Case 1.

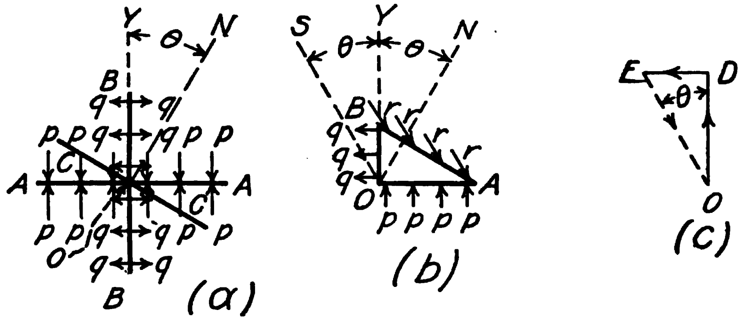


FIG. 4.

Now $OE^2 = DO^2 + DE^2$,

$= p.^2AB^2$, and $OE = p.AB$, as in Case 1,

but $OE = AB.r$,

and $r = p = q$

The stresses in Case 2 are *right shearing stresses.*

Case 3.—*Given the principal stresses of the same kind but having unequal intensities to determine the intensity and direction of the stress on a third plane.* If p and q represent the intensities of the forces on the planes A-A and B-B, respectively, acting through the point O, in (a), Fig. 5, p being the greater, we have by algebra

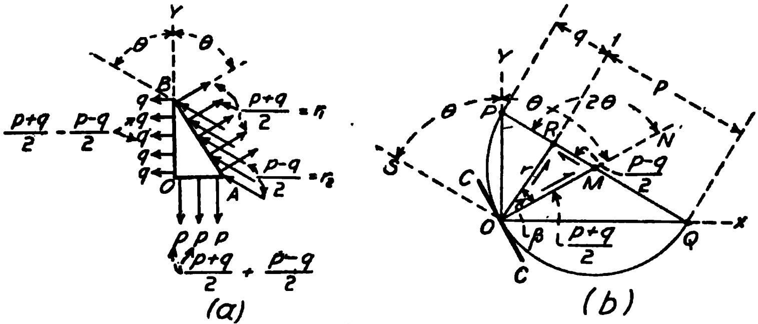
$$p = \frac{p+q}{2} + \frac{p-q}{2} \text{ an identity,}$$

$$\text{and } q = \frac{p+q}{2} - \frac{p-q}{2} \text{ an identity.}$$

Now we may look on the plane A-A as having two separate stresses equal to $\frac{p+q}{2}$ and $\frac{p-q}{2}$ of the same kind, while upon the plane B-B we will have two stresses $\frac{p+q}{2}$ and $-\frac{p-q}{2}$ of opposite kinds.

We may now group the stresses separately; thus the equal-like stresses $\frac{p+q}{2}$ and $\frac{p+q}{2}$ acting on the planes A-A and B-B, and the equal-unlike stresses $\frac{p-q}{2}$ and $-\frac{p-q}{2}$ acting on the same planes A-A and B-B, respectively.

Now the stress on the plane *C-C* for the equal-like stresses $\frac{p+q}{2}$ and $\frac{p+q}{2}$ is the force $r_1 = \frac{p+q}{2}$ acting normal to plane *C-C*. The stress on the plane *C-C* for the equal-unlike stresses $\frac{p-q}{2}$ and $-\frac{p-q}{2}$ will be a stress $r_2 = \frac{p-q}{2}$, and inclined at an angle θ to the line of principal stress. In (a), Fig. 5 we have the stresses acting as above and it is required to find the resultant of the stresses on the plane *C-C*.



(a) (b)

Fig. 5.

In (b) lay off $OM = \frac{p+q}{2} = r_1$, normal to *C-C*, and from *M* lay off $MR = \frac{p-q}{2} = r_2$, and parallel to $\frac{p-q}{2}$ in (a). Now $OR = r$ will be the resultant force in direction and intensity. From the construction it will be seen that in (b) $MP = MQ = OM = \frac{p+q}{2}$.

$$\text{Also } QR = MQ + MR,$$

$$= \frac{p+q}{2} + \frac{p-q}{2},$$

$$= p$$

$$PR = MP - MR,$$

$$= \frac{p+q}{2} - \frac{p-q}{2},$$

$$= q$$

$$\angle\, RMN = 2\theta$$

$$\angle\, ROM = \beta = \text{obliquity of } r.$$

The tangential component of r in Fig. 6 is

$$RT = MR \,.\, \sin RMT,$$

$$= \frac{p - q}{2} \sin 2\theta,$$

$$r_t = (p - q) \sin\theta \,.\, \cos\theta \tag{1}$$

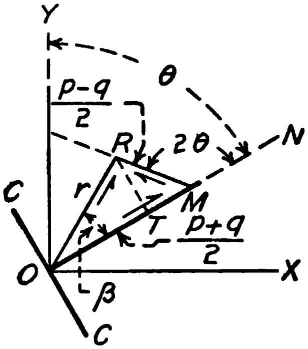


FIG. 6.

The normal component of r is

$$OT = OM - MT,$$

$$= OM + MR \,.\, \cos 2\theta,$$

$$= \frac{p+q}{2} + \frac{p-q}{2} (\cos^2\theta - \sin^2\theta),$$

$$= \frac{p+q}{2} (\sin^2\theta + \cos^2\theta) + \frac{p-q}{2} (\cos^2\theta - \sin^2\theta),$$

$$r_n = p \,.\, \cos^2\theta + q \,.\, \sin^2\theta \tag{2}$$

Note.—If t_n is the normal component of the stress on a plane *D-D*, a plane normal to *C-C*,

$$t_n = p.\cos^2(\theta + 90°) + q.\sin^2(\theta + 90°),$$

$$= p.\sin^2\theta + q.\cos^2\theta$$

Now adding r_n and t_n

$$t_n + r_n = p(\sin^2\theta + \cos^2\theta) + q(\sin^2\theta + \cos^2\theta) = p + q \qquad (3)$$

which proves that the sum of the normal stresses on any two rectangular planes is a constant, and is equal to the sum of the principal stresses.

Now as the plane *C-C* in (b), Fig. 5, moves through all angles the point *M* will describe a circle around *O*, with a radius $=\frac{p+q}{2}$, and *R* will describe a circle about *M*, with a radius $=\frac{p-q}{2}$, *OM* and *RM* keeping equally inclined to the vertical (direction of principal stress) on opposite sides of it. The locus of the point *R* is an ellipse, the semi-major axis being $OM + MR = p$, and the semi-minor axis being $OM - MR = q$. This is called the ellipse of stress for the point *O* within the body at the point *O*. Its principal axes are normal to the planes of principal stress, the semi-axes being equal to the principal stresses. The radii vectores OR, OR_1, etc., are the stresses on the planes at *O*, to which OM, OM_1, etc., are normals.

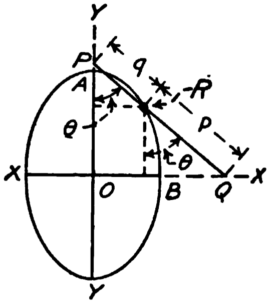


FIG. 7.

Equation of Ellipse of Stress.—The ordinary ellipsograph consists of a piece like *PRQ*, Fig. 7, whose extremities *P* and *Q* slide in two grooves *YOY* and *XOX*, respectively, at right angles to each other, while the point *R* traces an ellipse whose semi-axes are $PR = q$, and $RQ = p$.

Take coördinates of R as x and y,

$$\text{and } x = PR.\sin\theta = q.\sin\theta,$$

$$\frac{x}{q} = \sin\theta$$

$$y = RQ.\cos\theta = p.\cos\theta,$$

$$\frac{y}{p} = \cos\theta$$

Squaring and adding,

$$\frac{x^2}{q^2} + \frac{y^2}{p^2} = \sin^2\theta + \cos^2\theta = 1 \qquad (4)$$

Maximum value of $\beta = \phi$ the angle of obliquity of stress on plane C-C. It can be seen in Fig. 6 that β will be a maximum when MR is perpendicular to RO. For suppose triangle MOR constructed with angle ORM not a right angle, drop a perpendicular from M to OR at R', then $\sin\beta' = \frac{MR'}{OM}$. But MR' is less than MR and β' will be less than β.

In this case since angle $ORM = 90°$,

$$OR^2 = OM^2 - RM^2,$$

$$r^2 = \left(\frac{p+q}{2}\right)^2 - \left(\frac{p-q}{2}\right)^2,$$

$$= p.q,$$

$$\text{and } r = \sqrt{p.q},$$

$$\text{also } \sin\phi = \sin\beta = \frac{p-q}{p+q},$$

$$\text{and transposing } \frac{q}{p} = \frac{1 - \sin\phi}{1 + \sin\phi} \qquad (5)$$

$$\text{now } 2\theta = \text{angle } RMN, \text{ and}$$

$$\cos 2\theta = \cos RMN,$$

$$= -\cos ROM,$$

$$= -\frac{MR}{OM},$$

$$= -\frac{p-q}{p+q}$$

Now in a granular mass the particles will tend to slide on each other, the angle ϕ being the angle of internal friction in which the coefficient of friction is $\mu = \tan \phi$.

From equation (5) if p represents the vertical stress at any point in an unlimited, homogeneous, granular mass with a horizontal surface, the ratio of the horizontal component q to the vertical stress p will be

$$\frac{q}{p} = \frac{1 - \sin \phi}{1 + \sin \phi}$$

For a perfect fluid $\phi = 0$, and $p = q$, which we already know.

Angle of Repose.*—Loose earth will remain in equilibrium with its face at slopes whose inclinations to the horizontal are less than an angle ϕ, which is called the angle of repose. If piled at a greater slope, cohesion will hold the face at a greater slope than ϕ for a time, but the earth will soon crumble down, until the slopes do not exceed ϕ. In (a) Fig. 8, two troughs filled with earth are acted upon by the two forces P and Q. The forces P tend to press the earth together, while the forces Q tend to cause slipping. If Q is just sufficient to cause slipping, then the coefficient of friction will be $\mu = \frac{Q}{P}$. Now if the forces Q are omitted and the forces P are inclined at an angle θ, when the earth is just on the point of slipping angle θ will be equal to ϕ, the angle of repose and the coefficient of friction will be $\mu = \frac{P. \sin \phi}{P. \cos \phi} =$

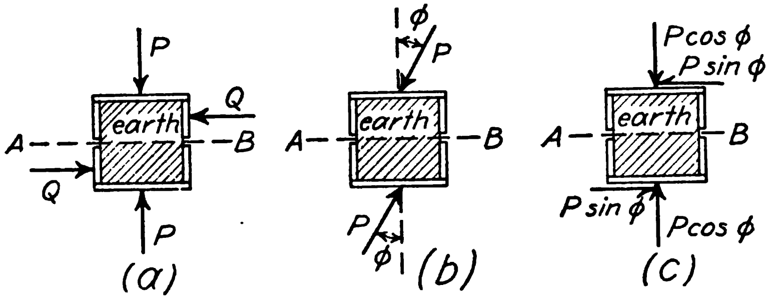


FIG. 8.

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* This is more properly called the angle of internal friction, see Table IX.

tan ϕ which gives the relation between the coefficient of friction μ, and the angle of repose ϕ.

Now in a homogeneous, unlimited, granular mass with a level surface, if p is the vertical and q is the horizontal stress at any point, then $\frac{q}{p}=\frac{1-\sin\beta}{1+\sin\beta}$, where β is the angle of obliquity of the resultant stress r. Now if there is to be equilibrium the obliquity, β, cannot be greater than ϕ, the angle of repose, and the greatest ratio between q and p will be $\frac{q}{p}=\frac{1-\sin\phi}{1+\sin\phi}$

Vertical Retaining Wall Without Surcharge.—In Fig. 9, a vertical wall supports a filling with a horizontal slope ($\delta=0$). Then from the preceding discussion the horizontal pressure at any point will be $q=w.y.\frac{1-\sin\phi}{1+\sin\phi}$, where y is the depth and w is the weight of a cubic foot of earth, and the total pressure will be

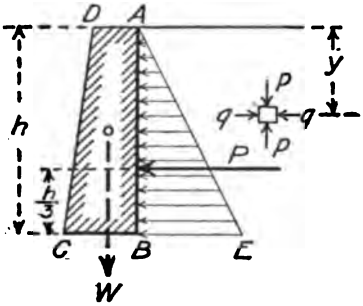

FIG. 9.

$$P=\tfrac{1}{2}w.h.^2\frac{1-\sin\phi}{1+\sin\phi} \qquad (6)$$

The stress q varies directly with the depth y, and the total stress P may be represented by the area of the triangle ABE, with the point of application of P at a point $\frac{1}{3}h$ above the base. The resultant stress P will be horizontal.

Vertical Retaining Wall With Surcharge.—In Fig. 10, the parallelopipedon is held in equilibrium by three forces: r vertical, s normal, and r' direction not yet determined. Now every point in a plane parallel to the sloping plane of surcharge must be in the same condition, which can only be the case when r' is parallel to the plane of surcharge (this may be proved by means of the ellipse of stress).

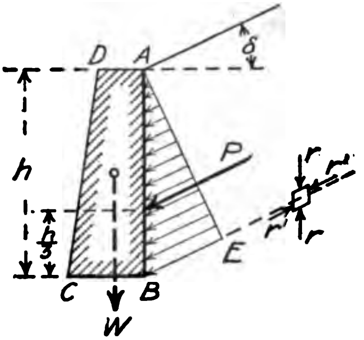

FIG. 10.

To find the ratio of the intensity of the conjugate stresses r and r' whose obliquity is δ, proceed as follows:

In Fig. 11, draw any line ON, draw OR making an angle δ with

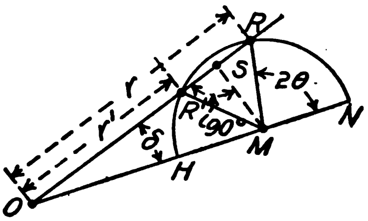

FIG. 11.

ON, lay off $OR = r$, and $OR' = r'$ and from S, the middle-point of RR' erect the perpendicular SM. Draw MR, MR', and arc $HR'RN$.

Now $OM = \frac{p+q}{2}$ and $MR = \frac{p-q}{2}$ as can be seen in Fig. 6.

$$\frac{OS}{OM} = \cos MOS, \text{ and solving}$$

$$OM = \frac{\frac{1}{2}(OR + OR')}{\cos\delta} = \frac{r + r'}{2\cos\delta}$$

But $OM = \frac{p+q}{2}$, and

$$\frac{p+q}{2} = \frac{r+r'}{2\cos\delta} \tag{a}$$

Also $MR^2 = MS^2 + RS^2$,

$$= (OM^2 - OS^2) + RS^2,$$

$$= \left(\frac{p+q}{2}\right)^2 - \left(\frac{r+r'}{2}\right)^2 + \left(\frac{r-r'}{2}\right)^2,$$

$$= \left(\frac{p+q}{2}\right)^2 - r.r',$$

$$= \left(\frac{r+r'}{2\cos\delta}\right)^2 - r.r' \tag{b}$$

Therefore, $\frac{p-q}{2} = \sqrt{\left(\frac{r+r'}{2\cos\delta}\right)^2 - r.r'}$ (c)

From Fig. 11, $\cos 2\theta = \frac{2r.\cos\delta - p - q}{p-q}$ (d)

Now squaring (a) and (c), and dividing (c) squared by (a) squared, we have $\left(\frac{p-q}{p+q}\right)^2 = 1 - \frac{4r.r.'\cos^2\delta}{(r+r')^2}$

But when the earth is just in equilibrium

$$\frac{p}{q} = \frac{1+\sin\phi}{1-\sin\phi}, \text{ or } \sin\phi = \frac{p-q}{p+q}$$

Therefore, $\sin^2\phi = 1 - \frac{4r.r.'\cos^2\delta}{(r+r')^2}$, and

$$1 - \sin^2\phi = \cos^2\phi = \frac{4r.r.'\cos^2\delta}{(r+r')^2}, \tag{e}$$

$$\frac{\cos^2\phi}{\cos^2\delta} = \frac{4r.r'}{(r+r')^2},$$

$$(r+r')^2 = 4r.r'\frac{\cos^2\delta}{\cos^2\phi}$$

Now subtract $4r.r'$ from both sides and

$$(r-r')^2 = 4r.r'\left(\frac{\cos^2\delta}{\cos^2\phi} - 1\right),$$

$$= 4r.r'\frac{\cos^2\delta - \cos^2\phi}{\cos^2\phi} \qquad \text{(f)}$$

Divide (f) by (e) and

$$\left(\frac{r-r'}{r+r'}\right)^2 = \frac{\cos^2\delta - \cos^2\phi}{\cos^2\phi}\cdot\frac{\cos^2\phi}{\cos^2\delta},$$

$$= \frac{\cos^2\delta - \cos^2\phi}{\cos^2\delta},$$

$$\text{and}\ \frac{r-r'}{r+r'} = \pm\sqrt{\frac{\cos^2\delta - \cos^2\phi}{\cos^2\delta}} \qquad \text{(g)}$$

Now by composition and division

$$\frac{r}{r'} = \frac{\cos\delta \pm \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta \mp \sqrt{\cos^2\delta - \cos^2\phi}} \qquad \text{(h)}$$

Now formula (h) represents both the active and the passive thrust at the point, the two stresses being equal in amount but opposite in direction. And since r' is less than r for the active forces, equilibrium of the wall will take place with the upper signs, and, reversing the fractions

$$\frac{r'}{r} = \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \qquad \text{(i)}$$

Now r is equal to $w.y.\cos\delta$, for the reason that the stress p is distributed over the area $a.\sec\delta$, and

$$r' = w.y.\cos\delta\,\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \qquad \text{(j)}$$

Now the maximum value of r' will be

$$r' = w.h.\cos\delta\,\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \qquad \text{(7)}$$

and

$$P = \tfrac{1}{2} w.h^2 . \cos\delta \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \qquad (8)$$

Equations (6) and (8) are commonly called Rankine's Method. As such the method is very unsatisfactory; the two cases above being only special cases of the general method which follows.

Inclined Retaining Wall.—The ellipse of stress can be used to determine the resultant pressure on an inclined retaining wall.* This solution determines the amount and direction of the resultant and leads to the same equations as deduced by Weyrauch by a much longer process.

In Fig. 12 a retaining wall inclined at the angle θ with the horizontal sustains the pressure of a filling with a surcharge δ. The angle of repose of the filling is ϕ.

Ellipse of Stress at Point A.—Through A draw AO parallel to the top surface, and at any convenient point O, in AO, draw OD normal to AO. With point O as a center and a radius OD describe an arc intersecting a vertical line OM at the point M. Draw OC making an angle ϕ with OD. At any point E in OG describe an arc which shall be tangent to OC and cuts the vertical OM at F. Draw EF, and through M draw MG parallel to EF. Bisect the angle DGM, and draw $R'G$. Through O draw OR' parallel to $R'G$. Now $OM.w$ is the stress, r, on the plane A-O, $OG = \frac{p+q}{2}$ and $GM = \frac{p-q}{2}$; OG and GM make equal angles with the principal axis OR'. If OG revolves around O, and GM around G, OG and GM always making equal angles with the principal axis OR', the ellipse of stress will be described as shown. To determine the pressure at the point A on the plane A-B, draw $OG' = OG$ at right angles to A-B. With G' as a center and a radius $OG' = OG$ describe an arc cutting the principal axis OR' at T. Draw TG'. With G' as a center and a radius $G'M' = GM$ describe an arc intersecting TG' at M'. Draw OM'. Then the pressure at A is $w.OM'$ acting in the direction $M'O$.

* For an application of this method to a problem see Fig. 27.

The construction may be made at the point A. The line of principal stress acts through A. Draw AK normal to $A\text{-}B$ and $= OG$; $QK = OG$; and $NK = GM$. Then $NA = OM'$, is in line of pressure

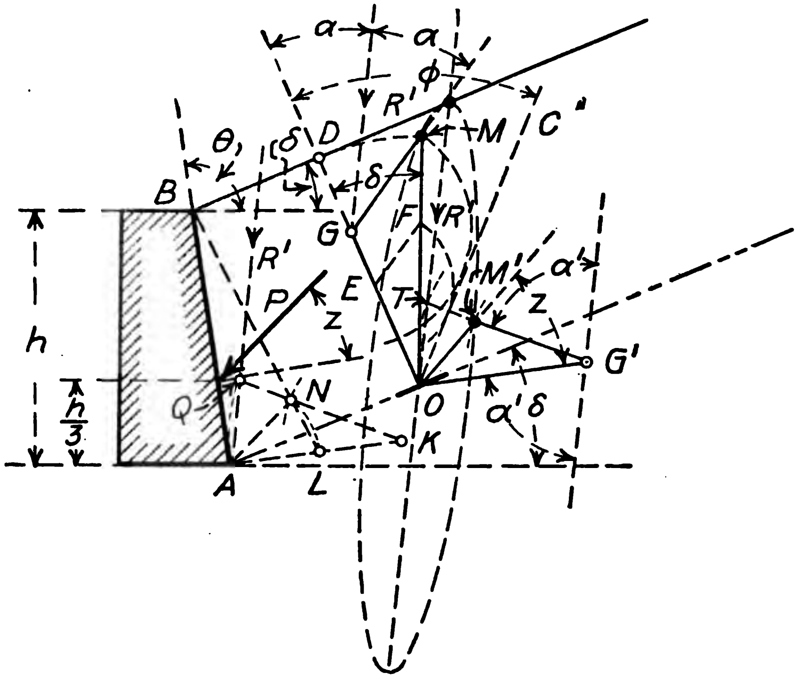

FIG. 12.

and the pressure at $A = w.AN$. With A as a center and a radius AN describe an arc and $AL = AN$. Then the resultant stress is $P =$ area triangle $ABL.w$.

The same results may be obtained directly from the discussion of the pressure on vertical walls as follows:

In Fig. 13 let $P_1 =$ pressure on a vertical wall EB as given by formula (8). P_1 acts parallel to the top slope and at a point $\frac{1}{3}BE$ above B. Let the weight of the triangle DBE be G, which acts through the center of gravity of the triangle and intersects P_1 at point O. Then P_2 the resultant of P_1 and G will be the resultant pressure at O; makes the

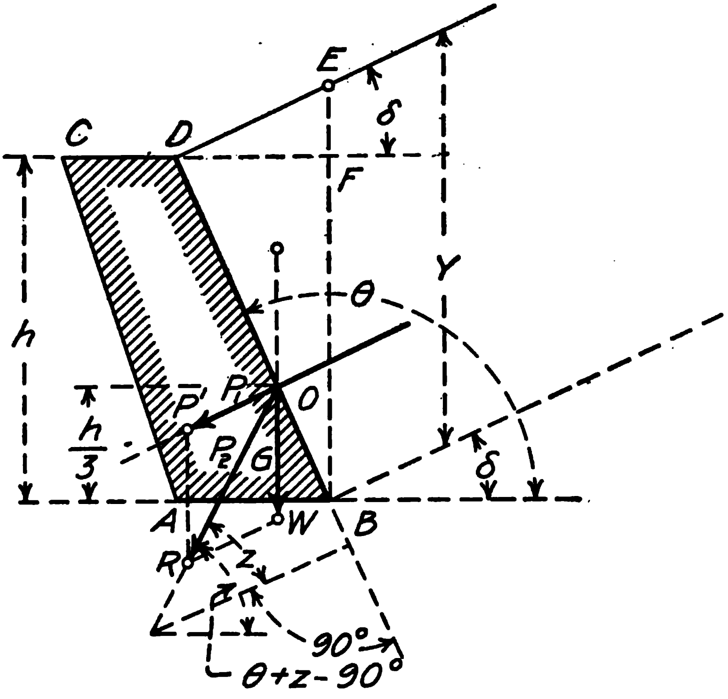

FIG. 13.

angle z with a normal to the back of the wall, and an angle $z + \theta - 90°$ with the horizontal.*

The algebraic equations for the values of P_2 and z are complicated and will not be given.†

Conclusion.—It will be seen that the direction of the resultant thrust on a vertical wall is parallel to the top plane surface, and that the pressure on an inclined plane may be calculated by the ellipse of stress. It can be easily proved that z, the angle between the direction of the resultant thrust and a normal to the plane, is never greater than ϕ, the angle of repose of the filling.

With a negative surcharge the maximum value of the resultant thrust consistent with stability occurs when the line of thrust is horizontal.

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* For practical applications of this method see Fig. 32 and Fig. 34.

† For algebraic equations see Howe's Retaining Walls.

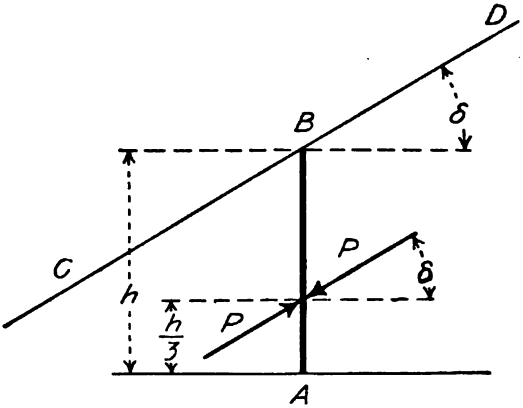


FIG. 14.

This can be shown as follows: In Fig. 14 the passive resistance *P* is equal to the active resultant *P* on the vertical plane *A*-*B*, and both are parallel to the slope of the filling. Now if the surcharge on the right is negative as in Fig. 15, the passive resistance on the left is still *P*

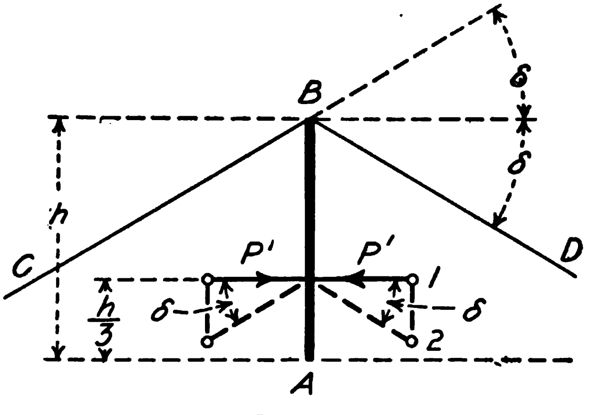


FIG. 15.

and is equal to the passive resistance *P* on the right. Now if the active resultants on both sides of the plane have an upward component, the plane under certain conditions would be forced out of the fill. The maximum value of the active resultant pressure must therefore occur when the line of action of *P* is horizontal and will be equal to *P′*.

Foundations in Earth.—*Case I.* Pressure uniform over the base of foundation as in Fig. 16, and equal p_0. When the superstructure has just stopped subsiding, and the earth on each side is just on the point of heaving up, p_0 will be a maximum and we will have

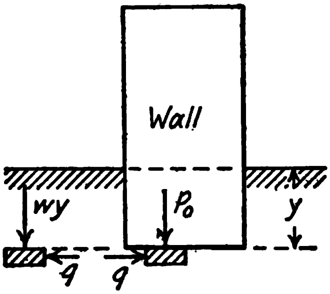


FIG. 16.

$$\frac{p_0}{q} = \frac{1 + \sin\phi}{1 - \sin\phi} \text{ or } q = p_0 \frac{1 - \sin\phi}{1 + \sin\phi}$$

At the same level outside of the foundation the horizontal pressure must be a maximum to be on the point of heaving, and

$$\frac{q}{w.y} = \frac{1 + \sin\phi}{1 - \sin\phi},$$

q being the active force and $w.y$ being the passive force.

Now eliminating q, we have

$$p_0 \leqq w.y \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2 \tag{9}$$

If h is the height of the wall and w' is the weight per cu. ft., then

$$w'h \leqq w.y \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2 \tag{10}$$

Case II.—Pressure varying uniformly from a maximum p_1 to a minimum p_2, as in Fig. 17.

By Case I

$$p_1 \leqq w.y \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2 \tag{11}$$

At the same level in the ground outside the foundation the minimum horizontal intensity is

$$q = w.y\,\frac{1 - \sin\phi}{1 + \sin\phi}$$

Now for equilibrium the pressure p_2 must produce a horizontal pressure not less than q. Therefore p_1 must not exceed the value given in (9) and p_2 must not be less than $w.y$.

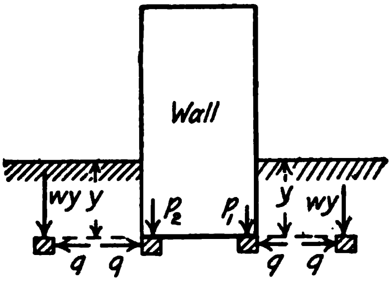


FIG. 17.

From the above discussion

$$\frac{p_1}{p_2} \leqq \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2 \qquad (12)$$

CHAPTER II.

COULOMB'S THEORY.

Introduction.—In this theory it is assumed that there is a wedge having the wall as one side and a plane called the plane of rupture as the other side, which exerts a maximum thrust on the wall. The plane of rupture lies between the angle of repose of the filling and the back of the wall. It may coincide with the plane of repose. For a wall without surcharge (horizontal surface back of the wall) and a vertical wall the plane of rupture bisects the angle between the plane of repose and the back of the wall. This theory does not determine the direction of the thrust, and leads to many other theories having assumed directions for the resultant pressure.

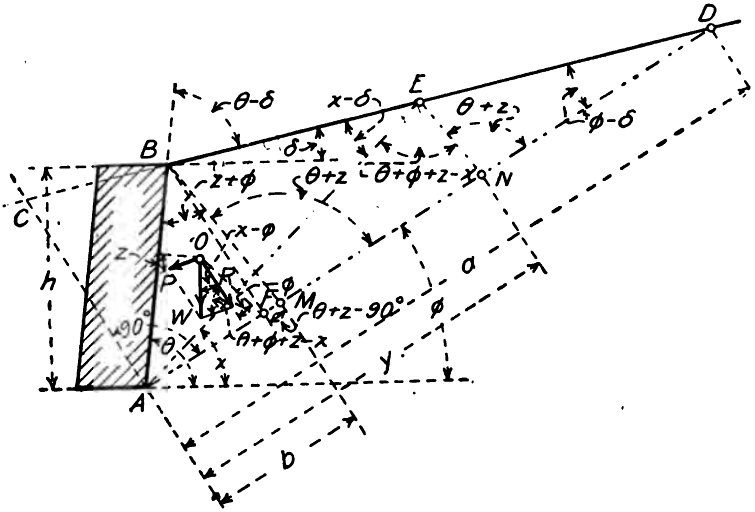


FIG. 18.

Algebraic Method.—In Fig. 18, the wall with a height h, slopes toward the earth, being inclined to the horizontal at an angle θ, and the earth has a surcharge with slope δ, which is not greater than ϕ, the angle of repose. It is required to find the pressure P against the retaining wall, it being assumed that the resultant pressure makes an angle z with the back of the wall.

It is assumed that the triangular prism of earth above some plane, the trace of which is the line AE, will produce the maximum pressure on the wall and on the earth below the plane, and that in turn the prism will be supported by the reactions of the wall and the earth. Let OW represent the weight of the prism ABE, the length of the prism being assumed equal to unity, let OP be the reaction of the wall, and OR be the reaction of the earth below.

Now the forces OW, OP, and OR will be concurrent and will be in equilibrium; OP and OR will therefore be components of OW. When the prism ABE is just on the point of moving OP will make an angle with a normal to the back of the wall equal to z (different authorities assume values of z from zero to ϕ', the angle of friction of earth on masonry, or ϕ, the angle of repose of earth); while OR will make an angle with the normal to the plane of rupture AE equal to ϕ. Let P represent the pressure OP against the wall, W represent the weight of the prism of earth and w the weight per cubic foot.

In the triangle OWR angle $WOR = x - \phi$, and angle $ORW = \theta + \phi + z - x$. Through E draw EN, making the angle $AEN = \theta + \phi + z - x$ with AE. Then the triangle AEN is similar to triangle ORW, and

$$\frac{P}{W} = \frac{EN}{AN}, \text{ and}$$

$$P = W\frac{EN}{AN}$$

But W equals w. *area triangle* $ABE = \frac{1}{2}w.AB.BE. \sin(\theta - \delta)$, and

$$P = \tfrac{1}{2}w.\sin(\theta - \delta)\frac{AB.BE.EN}{AN} \tag{13}$$

Now P varies with the angle x, and will have a maximum value for some value of x, which may be found by differentiating (13) and placing the result equal to zero.

Now draw BM parallel to EN, and in the similar triangles BDM and EDN,

$$EN : BM :: ND : MD, \text{ and}$$

$$EN = \frac{BM.ND}{MD}$$

$$BE : BD :: MN : MD, \text{ and}$$

$$BE = \frac{BD.MN}{MD}$$

Substituting in (13)

$$P = \tfrac{1}{2}w.\sin(\theta - \delta)\frac{AB.BM.BD}{MD^2}\left(\frac{MN.ND}{AN}\right) \qquad (14)$$

Where all the variables are in parenthesis. It will be seen that EN moves parallel to its present position as x varies. Now let $AN = y$, $AD = a$, and $AM = b$, and then

$$\frac{MN.ND}{AN} = \left(\frac{(y-b)(a-y)}{y}\right) \qquad (15)$$

Differentiating (15) and placing the result equal to zero; we have a maximum when $y = \sqrt{ab}$

Substituting in (14) we have

$$P = \tfrac{1}{2}w.\sin(\theta - \delta)\frac{AB.BM.BD}{MD^2}\frac{(a - \sqrt{ab})^2}{a} \qquad (16)$$

But $a = AD = AB\dfrac{\sin(\theta - \delta)}{\sin(\phi - \delta)}$,

$$MD = a - b,$$

$$BD = AB\frac{\sin(\theta - \phi)}{\sin(\phi - \delta)},$$

$$BM = AB\frac{\sin(\theta - \phi)}{\sin(\theta + z)}$$

Substituting in (16)

$$P = \tfrac{1}{2}w\frac{\sin^2(\theta - \phi)AB^2(a - \sqrt{ab})^2}{\sin(\theta + z)(a - b)^2},$$

$$= \tfrac{1}{2}w\frac{\sin^2(\theta - \phi)AB^2}{\sin(\theta + z)}\left(\frac{1}{1 + \sqrt{\frac{b}{a}}}\right)^2$$

Now

$$\frac{AM}{AB}=\frac{\sin(z+\phi)}{\sin(\theta+z)},$$

$$\frac{AB}{AD}=\frac{\sin(\phi-\delta)}{\sin(\theta-\delta)}, \text{ and}$$

$$\sqrt{\frac{AM}{AB}\cdot\frac{AB}{AD}}=\sqrt{\frac{b}{a}}$$

$$=\sqrt{\frac{\sin(z+\phi).\sin(\phi-\delta)}{\sin(\theta+z).\sin(\theta-\delta)}}$$

also, $AB=\frac{h}{\sin\theta}$, and finally

$$P=\tfrac{1}{2}w.h^2\frac{\sin^2(\theta-\phi)}{\sin^2\theta.\sin(\theta+z)\left(1+\sqrt{\frac{\sin(z+\phi).\sin(\phi-\delta)}{\sin(\theta+z).\sin(\theta-\delta)}}\right)^2} \qquad (17)$$

$$=\tfrac{1}{2}w.h^2.K \qquad (18)$$

which is the general formula for the pressure on a retaining wall.

Now if z in (17) is made equal to ϕ', the angle of repose of earth on the wall,

$$P=\tfrac{1}{2}w.h^2\frac{\sin^2(\theta-\phi)}{\sin^2\theta.\sin(\theta+\phi')\left(1+\sqrt{\frac{\sin(\phi+\phi').\sin(\phi-\delta)}{\sin(\theta+\phi').\sin(\theta-\delta)}}\right)^2} \qquad (19)$$

which is Cain's formula (43) in another form.

If z in (17) is made equal to δ, and θ made equal to 90°,

$$P=\tfrac{1}{2}w.h^2\frac{\cos^2\phi}{\cos\delta\left(1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\delta)}{\cos^2\delta}}\right)^2} \qquad (20)$$

which is Rankine's formula (8) in another form.

If z in (17) is made equal to zero,

$$P=\tfrac{1}{2}w.h^2\frac{\sin^2(\theta-\phi)}{\sin^2\theta\left(1+\sqrt{\frac{\sin\phi.\sin(\phi-\delta)}{\sin\theta.\sin(\theta-\delta)}}\right)^2} \qquad (21)$$

which gives the normal pressure on a wall.

If θ in (21) $= 90°$,

$$P = \tfrac{1}{2} w.h^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\dfrac{\sin \phi \,.\, \sin(\phi - \delta)}{\cos \delta}}\right)^2} \qquad (22)$$

If δ in (22) $= 0°$,

$$P = \tfrac{1}{2} w.h^2 \frac{\cos^2 \phi}{(1 + \sin \phi)^2},$$

$$= \tfrac{1}{2} w.h^2 \tan^2 (45° - \tfrac{1}{2}\phi) \qquad (23)$$

$$= \tfrac{1}{2} w.h^2 \frac{1 - \sin \phi}{1 + \sin \phi} \qquad (24)$$

which is Rankine's formula (6) for a vertical wall without surcharge.

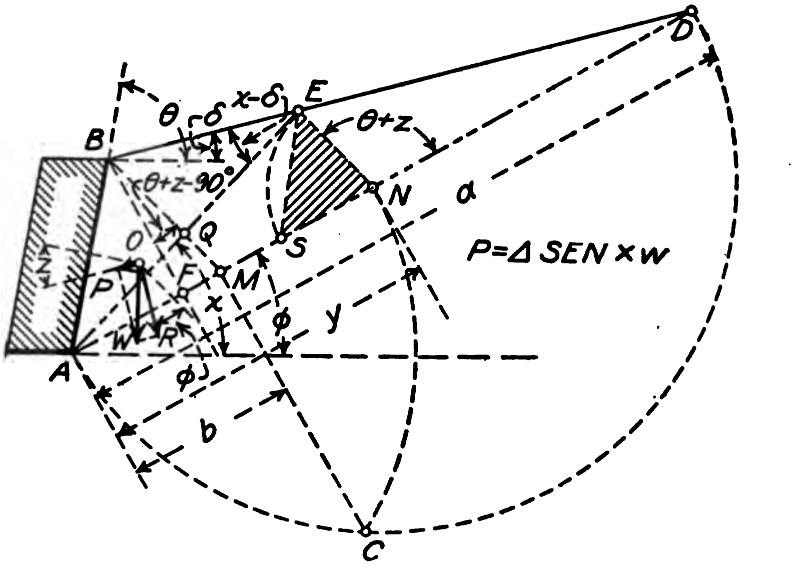


Fig. 19.

Graphic Method.—In Fig. 19 the retaining wall *AB* sustains the pressure of the filling with a surcharge δ, and an angle of repose ϕ. It is required to calculate the resultant pressure *P*.

In similar triangles *BMD* and *END*,

$$EN : BM :: ND : MD, \text{ and}$$

$$EN : BM :: (a - y) : (a - b),$$

$$EN = BM \frac{(a - y)}{(a - b)} \qquad (25)$$

In similar triangles ENA and QMA,

$$EN : QM :: y : b,$$

$$EN = QM \frac{y}{b}$$

Equating the two values of EN,

$$BM \frac{(a - y)}{(a - b)} = QM \frac{y}{b}, \text{ and}$$

$$BM : QM :: \frac{y}{b} : \frac{a - y}{a - b}, \text{ and by}$$

subtraction

$$BM - QM : BM :: \frac{y}{b} - \frac{a - y}{a - b} : \frac{y}{b},$$

$$BQ = BM \frac{a(y - b)}{b(a - b)} \frac{b}{y} = BM \frac{ay - y^2}{(a - b)y}$$

since $y = \sqrt{a.b}$, and $BQ = BM \dfrac{(a - y)}{(a - b)} = EN$ (25)

Therefore triangle $ABE =$ triangle AEN.

$$\text{Now } P = W \frac{\sin(x - \phi)}{\sin(\theta + \phi + z - x)}, \text{ and} \qquad (26)$$

$$W = \textit{area triangle } ABE.w,$$

$$= \textit{area triangle } AEN.w. \qquad (27)$$

Now in triangle AEN,

$$\frac{EN}{AN} = \frac{\sin(x - \phi)}{\sin(\theta + \phi + z - x)}, \qquad (28)$$

and area triangle $AEN = \frac{1}{2} EN \,.\, AN \,.\, \sin(\theta + z)$

From equations (26), (27), and (28)

$$P = \textit{area triangle } AEN \frac{EN}{AN} w,$$

$$= \tfrac{1}{2} EN^2 . \sin(\theta + z) . w \tag{29}$$

Now in triangle SEN, $SN = EN$, and

$$\textit{area } SEN = \tfrac{1}{2} EN^2 . \sin(\theta + z), \text{ and} \tag{30}$$

$$P = \textit{area triangle } SEN . w \tag{31}$$

The angle of rupture, x, and the resultant pressure, P, can therefore be calculated graphically as follows:

Through B in Fig. 19, draw BM making an angle with BF, the normal to AD, equal to $\theta + z - 90°$, the angle that P makes with the horizontal. With diameter AD, describe arc ACD. Draw MC normal to AD, and with A as a center and radius AC describe arc CN. Then $AN = y$, and $AM = b$. Draw EN parallel to BM. With N as a center and radius EN describe arc ES. Then AE is the plane of rupture, and $P = \Delta\, SEN . w$.

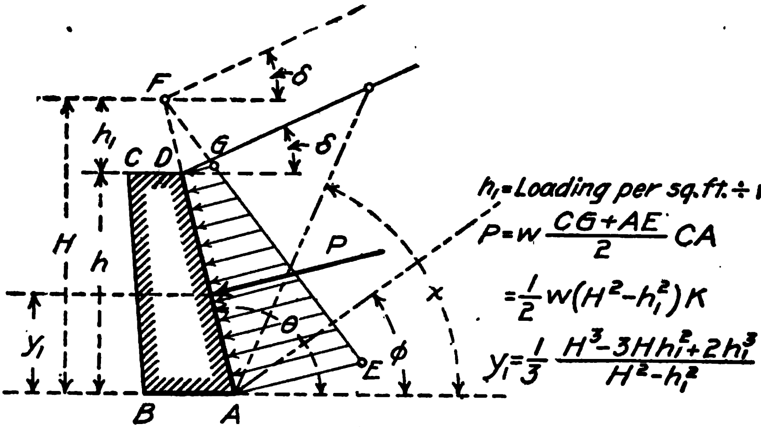

F
δ
δ
h_1
C
D
G
H
h
P
θ
x
φ
E
y_1
B
A
h_1 = Loading per sq. ft. ÷ W
$P = w \frac{CG + AE}{2} CA$
$= \frac{1}{2} w (H^2 - h_1^2) K$
$y_1 = \frac{1}{3} \frac{H^3 - 3Hh_1^2 + 2h_1^3}{H^2 - h_1^2}$

FIG. 20.

Wall With Loaded Filling.—In Fig. 20, the filling is loaded with a uniformly distributed load. Calculate h_1 by dividing the loading per square foot by w. Let $h + h_1 = H$. Then the resultant pressure for a wall with height H will be

$$P_2 = \tfrac{1}{2} w . H^2 . K \tag{32}$$

and the resultant pressure for a wall with height h_1, will be

$$P_1 = \tfrac{1}{2} w.h_1^2.K \tag{33}$$

The pressure on the wall AD will be

$$P = P_2 - P_1 = \tfrac{1}{2} w(H^2 - h_1^2)K \tag{34}$$

and the point of application is through the center of gravity of $ADGE$, which makes

$$y_1 = \tfrac{1}{3} \frac{H^3 - 3H.h_1^2 + 2h_1^3}{H^2 - h_1^2} \tag{35}$$

Surcharge, δ, Equal to Angle of Repose, ϕ.—Where $\delta = \phi$ and P is parallel to the top surface, the plane of rupture will coincide with the plane of repose, for in this case in Fig. 21 $a = \infty$ and $y = \infty$. To calculate P proceed as in Fig. 21, then

$$P = \Delta SEN.w$$

Where the wall is vertical, $\theta = 90°$, and $\delta = \phi$, the normal component of P for all values of z is constant. To prove this, substitute $\delta = \phi$ in (8), multiply by $\cos\phi$ and compare with (41).

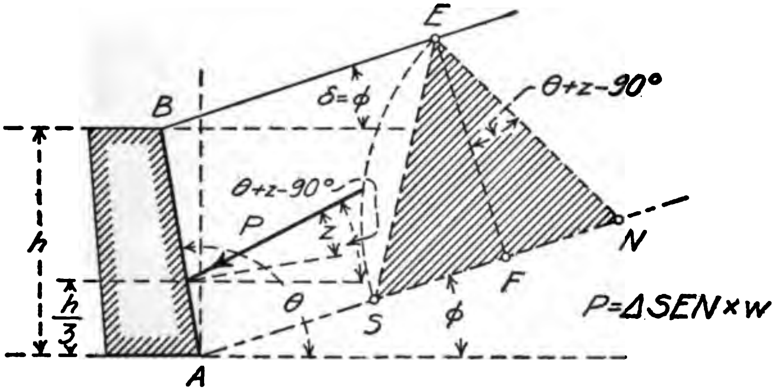


FIG. 21.

Surcharge, δ, Negative.—In this case the plane of repose and the plane of the surface of the filling meet at point D in Fig. 22. Draw BF and BM, making BF normal to AD and angle $FBM = \theta + z - 90°$. Then locate E graphically as in Fig. 19, and AE is the plane of rupture. Then the resultant stress is $P = \Delta SEN.w$.

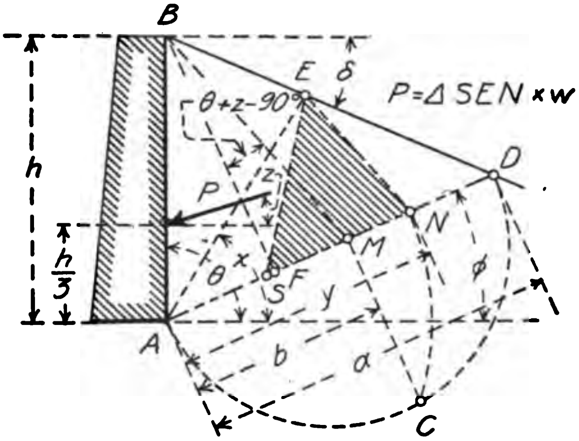


FIG. 22.

The maximum possible value of the active resultant pressure will occur when P is horizontal. The graphic solution for a vertical wall with P horizontal is given in Fig. 23.

Cain's Formulas.—Professor William Cain assumes that the angle z is equal to ϕ', the angle of friction of the filling on the back of the wall. By substituting in (17) we have for a

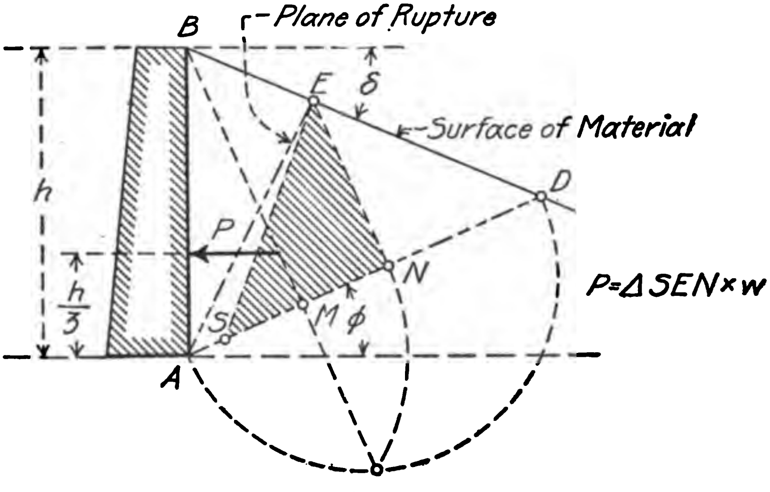


FIG. 23.

Vertical Wall With Level Surface, $\delta = 0$.

$$P = \tfrac{1}{2} w.h^2 \left(\frac{\cos\phi}{n+1}\right)^2 \frac{1}{\cos\phi'} \qquad (36)$$

where

$$n = \sqrt{\frac{\sin(\phi+\phi').\sin\phi}{\cos\phi'}}$$

If $\phi = \phi'$, then $n = \sqrt{2}\sin\phi$, and

$$P = \tfrac{1}{2} w.h^2 \frac{\cos\phi}{(1+\sin\phi\sqrt{2})^2} \qquad (37)$$

If $\phi' = 0$, then

$$P = \tfrac{1}{2} w.h^2 . \tan^2\left(45^\circ - \frac{\phi}{2}\right) \qquad (38)$$

Vertical Wall With Surcharge $= \delta$.

$$P = \tfrac{1}{2} w.h^2 \left(\frac{\cos\phi}{n+1}\right)^2 \frac{1}{\cos\phi'} \qquad (39)$$

where

$$n = \sqrt{\frac{\sin(\phi+\phi').\sin(\phi-\delta)}{\cos\phi'.\cos\delta}}$$

If $\delta = \phi$,

$$P = \tfrac{1}{2} w.h^2 \frac{\cos^2\phi}{\cos\phi'} \qquad (40)$$

If $\phi' = 0$, and $\delta = \phi$,

$$P = \tfrac{1}{2} w.h^2 . \cos^2\phi \qquad (41),$$

Inclined Wall With Horizontal Surface.

$$P = \tfrac{1}{2} w.h^2 \left(\frac{\sin(\theta-\phi)}{(n+1)\sin\theta}\right)^2 \frac{1}{\sin(\phi'+\theta)} \qquad (42)$$

where

$$n = \sqrt{\frac{\sin(\phi+\phi').\sin\phi}{\sin(\phi'+\theta).\sin\theta}}$$

Inclined Wall With Surcharge $= \delta$.

$$P = \tfrac{1}{2} w.h^2 \left(\frac{\sin(\theta-\phi)}{(n+1).\sin\theta}\right)^2 \frac{1}{\sin(\phi'+\theta)} \qquad (43)$$

where

$$n = \sqrt{\frac{\sin(\phi+\phi').\sin(\phi-\delta)}{\sin(\phi'+\theta).\sin(\theta-\delta)}}$$

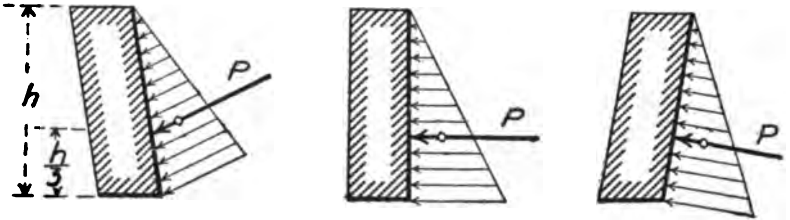
CHAPTER III.

DESIGN OF MASONRY RETAINING WALLS.

Center of Pressure.—It will be seen that for all cases the resultant pressure on the back of the wall will be given by the formula

$$P = \tfrac{1}{2}w.h^2.K \tag{18}$$

Where K is the ratio of the horizontal to the vertical pressure, and is independent of the weight of the filling or the height of the wall, and depends upon the inclination of the back of the wall, θ, the angle of repose, ϕ, the angle of surcharge, δ, and the angle that the resultant thrust makes with a normal to the back of the wall, z. For any particular case, K will be a constant and the unit pressure at any point will vary as the height h. In Fig. 24, the resultant pressure, P, is represented by the area of the shaded triangles, and the center of pressure will be at $\tfrac{1}{3}h$ from the base, the same as for fluid pressure.



(a) (b) (c)

FIG. 24.

Experiments made with models have shown that the center of pressure $= \tfrac{1}{3}h$ is sometimes exceeded. Goodrich recommends that for walls from 6 to 10 feet the center of pressure be taken at $0.4h$, and above 10 feet the center of pressure be taken at $\tfrac{1}{3}h$ above the base.* Scheffler assumed that the center of pressure is at $0.4h$.

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* Trans. Am. Soc. C. E., Vol. LIII, p. 295; also see Chapter V.

It should be remembered that experiments have been made on small models and are liable to considerable error. It would appear reasonable with our present knowledge to always take the center of pressure at a point $\frac{1}{3}h$ above the base.

Stability of Retaining Walls.—A retaining wall must be stable (1) against overturning, (2) against sliding, and (3) against crushing the masonry of the foundation.

1. *Overturning.*—In Fig. 25, let P, represented by OP', be the resultant pressure of the earth, and W, represented by OW, be the weight of the wall acting through its center of gravity. Then E, represented by OR, will be the resultant pressure tending to overturn the wall.

Draw OS through the point A. For this condition the wall should be on the point of overturning, and the factor of safety against overturning would be unity. The factor of safety for $E = OR$ will be

$$f_0 = \frac{SW}{RW} \tag{44}$$

2. *Sliding.*—In Fig. 25 construct the angle $H1G$ equal to ϕ', the angle of friction of the masonry on the foundation. Now if E passes through 1, and takes the direction OQ, the wall will be on the point of sliding, and the factor of safety against sliding, f_s, will be unity. For $E = OR$, the factor of safety against sliding will be

$$f_s = \frac{QM'}{RM} \tag{45}$$

If the resultant at 3 be resolved into a vertical force F, and a horizontal force H (not shown in Fig. 25), then

$$f = \frac{F}{H} \tan \phi' \tag{46}$$

which will give the same result as above.

Retaining walls seldom fail by sliding.

3. *Crushing.*—In Fig. 25 the load on the foundation will be due to

a vertical force F, which produces a uniform stress $p_1 = \frac{F}{d}$ over the area of the base, and a bending moment $= F.b$, which produces compression, p_2, on the front and tension, p_2, on the back of the foundation.

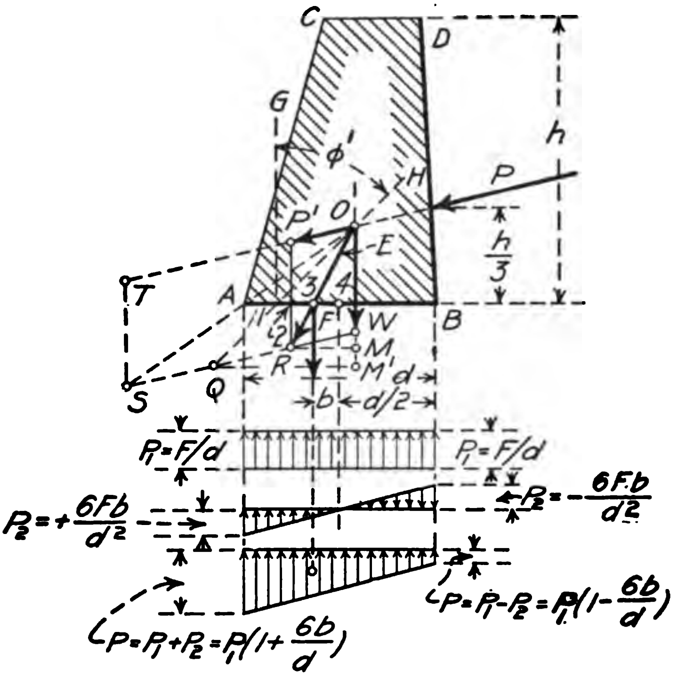

FIG. 25.

The sum of the tensile stresses due to bending must equal the sum of the compressive stresses, $= \frac{1}{4} p_2 d$. These stresses act as a couple through the centers of gravity of the stress triangles on each side, and the resisting moment is

$$M' = \tfrac{1}{4} p_2.d.\tfrac{2}{3} d = \tfrac{1}{6} p_2.d^2 \qquad (47)$$

But the resisting movement equals the overturning moment, and

$$\tfrac{1}{6} p_2.d^2 = F.b, \text{ and}$$

$$p_2 = \pm \frac{6F.b}{d^2} \qquad (48)$$

The total stress on the foundation then is

$$p = p_1 \pm p_2 = p_1 \left(1 \pm \frac{6b}{d}\right) \tag{49}$$

Now if $b = \frac{1}{6}d$, we will have

$$p = 2p_1, \text{ or } 0.$$

In order therefore that there be no tension, or that the compression never exceed twice the average stress the resultant should never strike outside the middle third of the base.

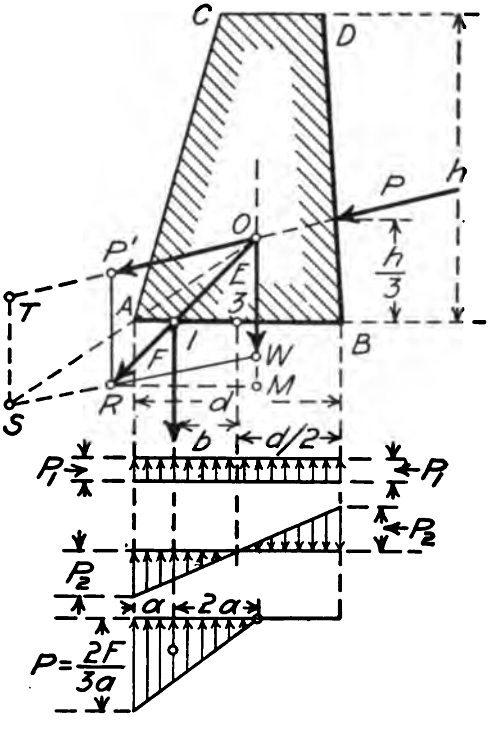

FIG. 26.

If the resultant strike outside of the middle third of a wall in which the masonry can take no tension, the load will all be taken by compression and can be calculated as follows:

In Fig. 26 the resultant F will pass through the center of gravity of the stress diagram, and will equal the area of the diagram.

$$F = \frac{3}{2} p.a, \text{ and}$$

$$p = \tfrac{2}{3} \frac{F}{a} \tag{50}$$

which gives a larger value of p than would be given if the masonry could take tension.

General Principles of Design.—In calculating the thrust on a retaining wall great care must be exercised in selecting the proper value of ϕ and the conditions of surcharge. It will be seen from the preceding discussion that the value of the thrust increases very rapidly as ϕ decreases, and as the surcharge δ increases. Where the wall is to sustain a railway or similar embankment, a proper allowance must be made for surcharge.

The filling back of the wall should be deposited and tamped in approximately horizontal layers or in layers sloping back from the wall, and a layer of sand, gravel or other porous material should be deposited between the fill and the wall to drain the fill downwards. To insure drainage of the filling, drains should be provided back of the footing and " weep-holes " should be provided in the body of the wall at frequent intervals to allow the water to pass through. The filling in front of the wall should also be carefully drained.

Economical Section.—The most economical section for a masonry retaining wall is obtained when the back slopes toward the filling. In cold localities, however, this form of section may be displaced by heaving due to the action of frost, and it is usual to build retaining walls with a very slight batter forwards. In any case the batter on the front of the wall should be greater than on the back in order that the center of gravity of the wall may be as near the back of the foundation as possible.

In order to insure that the frost will not heave the foundation it is usual to provide from 2½ to 5 feet of filling in front of the wall. To prevent cracks due to expansion and contraction, expansion joints are provided in long walls at intervals of from 30 to 50 feet.

The design of masonry retaining walls will be illustrated by an example.

Problem 1.—Design a masonry retaining wall of sandstone weighing 150 lbs. per cu. ft., having a height of 18 feet and a top width of $2\frac{1}{2}$ feet, to retain a bank of sand sloping upwards at an angle of 22° 30′. Sand weighs 100 lbs. per cu. ft., and has an angle of repose $\phi = 37° 30'$.

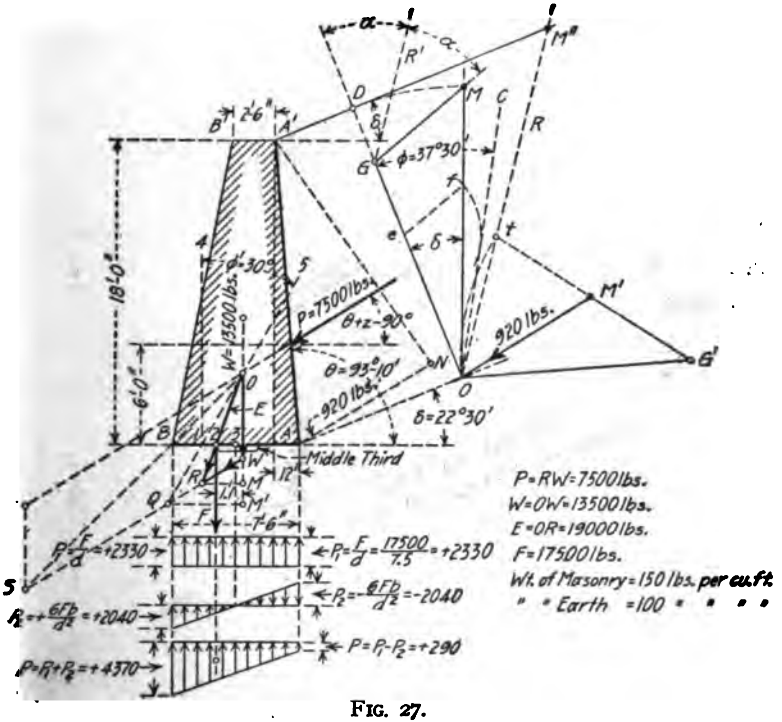

FIG. 27.

Solution.—The back of the wall in Fig. 27 will be assumed to batter $\frac{3}{4}$ inch to the foot, and the foundation will be assumed as 7 ft. 6 ins., which is approximately $\frac{4}{10}$ the height.

To calculate the resultant pressure P, proceed as follows:* Draw AO parallel to the surcharge $A'M''$, and at any convenient point O in

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* This problem is solved by means of the Ellipse of Stress, Fig. 12. The methods used in Fig. 32 and Fig. 34 will give the same result.

AO draw OD at right angles to AO. Draw OM vertical and locate M by striking the arc DM with O as a center. Draw OC making an angle ϕ with OD. At any point e in OD describe an arc tangent to OC and cutting OM at f. Through M draw MG parallel to ef. Bisect angle DGM, and through O draw OR parallel to GR'. Then OR is the principal axis of the ellipse of stress and OM'' is the maximum stress that can occur in the filling. To calculate the maximum stress at A, draw OG' at right angles to the back of the wall A-A', and make $OG' = OG$, with G' as the center and OG' as a radius describe an arc cutting the principal axis OR at t. Draw $G't$, and with G' as a center and a radius equal to GM describe an arc cutting $G't$ at M'. Then $M'O$ acting as shown is the intensity of the stress at A. The resultant pressure P is equal to the area of the stress triangle $AA'N$, equals 7,500 lbs., and acts on A-A' at $\frac{1}{3}$ the height of the wall.

The weight of the masonry is $W = 13{,}500$ lbs., acting through the center of gravity of the wall. The resultant of W and R is $E = OR = 19{,}000$ lbs.

The resultant E cuts the foundation at a point 1.1 ft. from the center, which is within the middle third. The vertical component of E is $F = 17{,}500$ lbs.

1. *Stability Against Overturning.*—Through B draw OS and produce RW to S. Then the factor of safety against overturning is $f_0 = \frac{SW}{RW} = 5$. If E passed through B the wall would be on the point of overturning and f_0 would equal unity.

2. *Stability Against Sliding.*—The coefficient of friction of the masonry on the footing will be taken as 0.57 and $\phi' = 30°$. Through O draw $5Q$, cutting base of wall $4B$ at 1, and making an angle $\phi' = 30°$ with 1–4. Then factor of safety against sliding will be

$$f_s = \frac{QM'}{RM} = 1.9$$

This is ample as the resistance of the filling in front of the toe will increase the factor of safety against sliding. The factor of safety against sliding at any point above the base can be determined in a similar manner, but will be much greater on account of the greater value of ϕ' for masonry on masonry.

3. *Stability Against Crushing.*—In Fig. 27 the direct pressure will be $p_1 = \frac{F}{d} = 17{,}500 \div 7.5 = 2{,}330$ lbs. per sq. ft.

The pressure due to bending moment will be

$$p_2 = \pm \frac{6Fb}{d^2} = \pm \frac{6 \times 17{,}500 \times 1.1}{56.25}$$

$$= \pm 2{,}040 \text{ lbs. per sq. ft.}$$

and the maximum pressure will be $p = 2{,}330 + 2{,}040 = + 4{,}370$ lbs., while the minimum pressure will be $p = 2{,}330 - 2{,}040 = + 290$ lbs. per sq. ft. By comparing with the allowable pressures in Table III it will be seen that the pressure is a safe one for a good sand and gravel or clay foundation.

4. *Depth of Filling in Front of Wall.*—From formula (9) it will be seen that to prevent heaving we must have

$$p \leqq w.y\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2 \qquad (9)$$

From which, when $w = 100$ lbs., $\phi = 37°\ 30'$ and $p = 4{,}370$ lbs. per sq. ft., $y \geqq 2.6$ ft. Which is less than the distance usually required to be safe from frost action.

The width of the top is about as small as is allowable where there is frost action. By spreading the footing the pressure can be decreased or the section may be slightly decreased.

The filling should be deposited and tamped in nearly horizontal layers, while a porous layer of gravel or similar material should be deposited just behind the wall to carry the moisture to the bottom of the wall, where a suitable drain should be provided with "weep-holes" leading through the wall at intervals of from 30 to 50 feet.

Empirical Method.—Take a rectangular masonry retaining wall as in Fig. 28. The masonry weighs 150 lbs. per cubic foot, the filling weighs 100 lbs. per cubic foot, and the ratio of the lateral to the vertical pressure will be taken as four-tenths.*

Then the resultant pressure is

$$P=\tfrac{4}{10}\cdot\frac{100h^2}{2}=20h^2$$

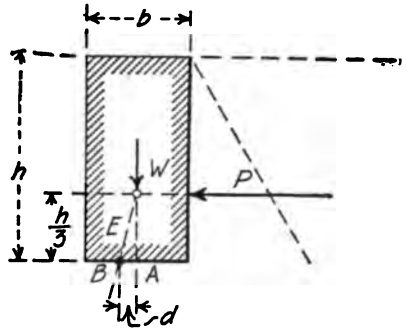

Fig. 28.

Total weight of wall is

$$W=150b.h.$$

Now in order that the maximum pressure be not greater than twice the average, and that there be no tension on the back side of the foundations, d must not be greater than $\frac{b}{6}$.

Taking moments about A we have

$$W.d-P\frac{h}{3}=0$$

$$150b.h.\frac{b}{6}-20h^2.\frac{h}{3}=0$$

$$15b^2=4h^2$$

$$b=h\sqrt{\tfrac{4}{15}}=0.52h=\frac{h}{2}\text{ (approx.)}$$

Trautwine's Rules.—Trautwine gives the following ratio of width of foundation b to height h, when the earth backing is loosely placed.

Wall of cut stone, or of first-class ranged rubble, in mortar $b=.35h$.

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* See Table IX for angles of repose and ratios of lateral to vertical pressures.

Wall of common scabbled mortar-rubble, or brick $b = .40h$.

Wall of well scabbled dry rubble $b = .50h$.

A mixture of sand or earth, with or without a large proportion of round bowlders, paving pebbles, etc., will weigh more and the thickness of the wall should be increased about one-eighth to one-sixth part.

Data.—The coefficients of friction of various materials are given in Table I. The conditions of surface and amount of moisture cause wide variations in the coefficients.

TABLE I.

COEFFICIENTS OF FRICTION.

Materials.	Coefficients.	Materials.	Coefficients.
Dry masonry on dry masonry...	0.6 to 0.7	Masonry on dry clay............	0.5 to 0.6
Masonry on masonry with wet mortar...........................	0.75	Masonry on moist clay.	0.33
Timber on stone	0.4	Earth on earth.	0.25 to 1.0
Iron on stone.	0.3 to 0.7	Hard brick on hard brick.......	0.7
Timber on timber	0.2 to 0.5	Concrete blocks on concrete blocks.............................	0.65

Approximate values of angle of repose, ϕ, are given in Table II.

TABLE II.

ANGLES OF REPOSE, ϕ, FOR MATERIALS.*

Materials.	ϕ	Materials.	ϕ
Earth, loam	30° to 45°	Clay..........................	25° to 45°
Sand, dry......................	25° to 35°	Gravel.......	30° to 40°
Sand, moist....................	30° to 45°	Cinders	25° to 40°
Sand, wet.......................	15° to 30°	Coke...........................	30° to 45°

Allowable pressures on foundations to be used in the absence of more definite data are given in Table III.

TABLE III.

ALLOWABLE PRESSURE ON FOUNDATIONS.

Material.	Pressure in Tons per Sq. Ft.
Soft clay..	1 to 2
Ordinary clay and dry sand mixed with clay........................	2 to 3
Dry sand and clay.........................	3 to 4
Hard clay and firm, coarse sand..	4 to 6
Firm, coarse sand and gravel.........	6 to 8
Bed rock..	15 and up.

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* Also see Table IX.

The allowable pressures on masonry are given in Table IV.

TABLE IV.

ALLOWABLE PRESSURE ON MASONRY.

Materials.	Pressure in Tons per Sq. Ft.
Common brick, Portland cement mortar	12
Paving brick, Portland cement mortar	15
Rubble masonry, Portland cement mortar	12
Sandstone, first class masonry	20
Limestone, " " "	25
Granite, " " "	30
Portland cement concrete, 1-2-4.	25
" " " 1-3-6	20

The weights, specific gravities and crushing strengths of masonry are given in Table V.

TABLE V.

WEIGHT, SPECIFIC GRAVITY AND CRUSHING STRENGTH OF MASONRY

Materials.	Weight in Pounds per Cubic Foot.	Specific Gravity.	Crushing Strength in Pounds per Square Inch
Sandstone	150	2.4	4,000 to 15,000
Limestone	160	2.6	6,000 to 20,000
Trap	180	2.9	19,000 to 33,000
Marble	165	2.7	8,000 to 20,000
Granite	165	2.7	8,000 to 20,000
Paving brick, Portland cement	150	2.4	2,000 to 6,000
Stone concrete, Portland cement	140 to 150	2.2 to 2.4	2,500 to 4,000
Cinder concrete, Portland cement	112	1.8	1,000 to 2,500

The average weights of materials are given in Table VI.

TABLE VI.

WEIGHT OF DIFFERENT MATERIALS.

Materials.	Wt. per Cu. Ft. Lbs.	Materials.	Wt. per Cu. Ft. Lbs.
Loam, loose.	75 to 90	Sand, wet	110 to 120
Loam, rammed	90 to 100	Gravel	120 to 135
Sand, dry	90 to 110	Soft flowing mud	105 to 120

For definitions of terms used in masonry construction, and for specifications for stone masonry, see Appendix I.

CHAPTER IV.

REINFORCED CONCRETE RETAINING WALLS.

Introduction.—Concrete reinforced with steel or iron has been extensively used in retaining wall construction. Retaining walls of reinforced concrete may be built (1) with a spreading base and a thin vertical section, or (2) may be built with a thin vertical section strengthened by buttresses or counterforts. The principal advantage of a reinforced concrete wall over a plain concrete wall is in the saving of material in substituting a thin reinforced wall for the gravity section and in substituting earth filling on the back of the footing in place of the expensive concrete. Before taking up the design of reinforced concrete retaining walls it will be necessary to discuss very briefly the theory of reinforced concrete.

THEORY OF REINFORCED CONCRETE.

CONCRETE.—Concrete is fabricated by thoroughly mixing cement, sand, and gravel or broken stone, together with sufficient water to produce a mixture of the proper consistency. The mixture is immediately deposited in molds and is well tamped. When concrete is reinforced with steel it is mixed with more water than when it is used plain.

Strength.—The strength of concrete varies with the proportions of the mixture, the quality of the cement and other ingredients, the method of mixing and depositing and the age. The following may be taken as average values. The cement is assumed to be a first-class American Portland, the sand a clean, moist, bank sand, and the stone is equal in quality to a first-class limestone and has about 45 per cent voids. Gravel concrete may be taken as from 75 to 80 per cent of the strength of broken stone concrete.

Compressive Strength.—Average values for the compressive strengths of Portland cement concrete of different proportions are given in Table VII.

TABLE VII.

COMPRESSIVE STRENGTH OF PORTLAND CEMENT CONCRETE IN LBS. PER SQUARE INCH.

Proportions.	Age, 1 Month.	Age, 6 Months.
1 cement, 2 sand, 4 broken stone	2,440	3,300
1 cement, 2 sand, 5 broken stone	2,350	3,180
1 cement, 3 sand, 5 broken stone	2,030	2,740
1 cement, 3 sand, 6 broken stone	1,950	2,630
1 cement, 3 sand, 8 broken stone	1,800	2,400
1 cement, 4 sand, 8 broken stone	1,570	2,120

Tensile Strength.—The tensile strength is difficult to determine by direct experiment, and is usually taken as one-tenth of the compressive strength. With factor of safety of 4 to 6 the safe tensile stresses will vary from 80 to 40 lbs. per sq. in.

To obtain safe working stresses use factor of safety of from 4 to 6. Working stresses of 500 lbs. per sq. in. for compression and 50 lbs. per sq. in. for tension are very commonly used for good 1–3–6 Portland cement concrete.

Modulus of Elasticity.—The modulus of elasticity of concrete is not a constant quantity as is the modulus of elasticity of steel, but varies with the proportions and ingredients, and with the stress in the concrete.

The following values have been proposed for use in reinforced concrete design where wet Portland cement concrete is used.

TABLE VIII.

MODULUS OF ELASTICITY OF PORTLAND CEMENT CONCRETE.

Proportions.		Modulus of Elasticity, Pounds per Square Inch.
	1 : 1½ : 3	4,000,000
	1 : 2 : 4	3,000,000
Broken stone or gravel concretes	1 : 2½ : 5	2,500,000
	1 : 3 : 6	2,000,000
	1 : 4 : 8	1,500,000
Cinder concrete	1 : 2 : 5	850,000

Shear.—Very few experiments have been made to determine the shearing strength of concrete. Recent tests would appear to show that the shearing strength of concrete is from 60 to 80 per cent of the compressive strength.

Adhesion to Steel Rods.—Where the yield point of the rod is not exceeded the adhesion of smooth rods to concrete varies from about 275 to 700 pounds per square inch of surface in contact. Professor W. K. Hatt, with seven-sixteenths-inch and five-eighths-inch rods imbedded 6 and 6.4 inches in 1–2–4 Portland cement concrete, obtained a strength at about 34 days of 636 and 756 pounds per square inch in contact. He found slipping friction after adhesion had been overcome to be 50 to 70 per cent of the adhesion. Deformed rods have a greater adhesion than plain rods. Experience would show that deformed rods should be imbedded in concrete not less than 50 diameters and plain rods not less than 80 diameters in order that the tensile strength of the rod be developed.

Tests of Reinforced Concrete Beams.—Many tests have been made to determine the strength and behavior of reinforced concrete beams. The tests by Professor A. N. Talbot, recorded in University of Illinois Engineering Experiment Station Bulletin, Nos. 1 and 4, are very valuable and instructive.

Professor Talbot among other conclusions draws the following from his tests of 1904:

"1. The composite structure acts as a true combination of steel and concrete in flexure during the first or preliminary stage, and this stage lasts until the steel is stressed to say 3,000 lbs. per sq. in., and the lower surface of the concrete is elongated nearly $\frac{1}{10000}$ of its length.

"2. During the second or readjustment stage there is a marked change in distribution of stresses, the neutral axis rises, the concrete loses part of its tensional value, and tensile stresses formerly taken by the concrete are transferred to the steel. During this stage minute cracks probably exist, quite well distributed and not easily detected.

"3. In the third or straight-line stage the neutral axis remains nearly stationary in position and the concrete gradually loses more of its tensional value. Visible cracks appear and gradually grow more distinct, though no change in the character of the load-deformation diagram results. It would seem probable that at these cracks the stress in the steel is more than is indicated by the average deformation for the full gage length.

"4. In beams with the metal reinforcement small enough in amount not to develop the full compression strength of the concrete the maximum load is reached or nearly reached when the metal is stretched to its yield point, and in calculating the resisting moment the tensional value of the concrete is here negligible, and the load at the yield point of the metal may well be considered the full strength of the beam. The resisting moment at this point may well be calculated by multiplying the stress in the total steel area by the distance to the center of gravity of compressive stresses.

"5. So far as strength of the beam is concerned, the load when the steel is stressed to its elastic limit seems the proper basis for the factor of safety and working load.

"6. So far as strength of beam is concerned, steel having a high elastic limit is advantageous, it being assumed that there is sufficient provision against the slipping of rods and shearing failures.

"7. The determination of the limit of reinforcement which may properly be used with different mixtures and grades of concrete may best be decided by experiments on beams made to determine this. For the 1 : 3 : 6 concrete used, reinforcement as high as one and one-half per cent for steel of 33,000 lbs. per sq. in. elastic limit and one per cent for steel of 55,000 lbs. per sq. in. elastic limit may be used without developing the full compression strength of the concrete."

The tests of 1905 confirm the preceding conclusions and in addition Professor Talbot among others draws the following conclusions:

"8. Beams which failed by diagonal tension developed an average vertical shear of 123 lbs. per sq. in. as calculated by formula (60). The diagonal tensile stress corresponding to the shearing stress may be considered to be the tensile strength of the concrete. These results have a bearing upon the importance of web reinforcement.

"9. Beams reinforced with tool steel failed by slipping of the rods with 133 lbs. per sq. in. as the average bond stress developed. The special tests to determine the bond resistance of tool steel rods averaged 149 lbs. per sq. in. In the beams reinforced with steel placed horizontally there was no evidence of slip although in one beam a bond stress of 193 lbs. was developed."

Resistance of Reinforced Concrete Beams to Flexure.—Reinforced concrete beams may fail by (1) tension of steel; (2) compression of concrete; (3) shear of concrete; (4) bond or slip of rods, (5) diagonal tension of concrete; (6) miscellaneous methods, like the splitting of concrete away from the rods, crushing of bearings, etc. A given beam is not liable to fail by all the methods. Only simple beams reinforced on the tension side will be considered.

From the preceding discussion it has seemed best to the author to take the third stage or straight line stage referred to by Professor Talbot as the basis for working formulas.

This theory assumes the following principles:

1. A plane section before bending remains a plane section after bending.
2. Tension is carried entirely by the steel.
3. No initial tension or compression in the steel.
4. Perfect adhesion of concrete to the steel.
5. Constant modulus of elasticity of concrete, as in the case of simple beams of steel.
6. The unit deformation in any horizontal fiber varies directly as its distance from the neutral axis.
7. In any cross-section of a beam the sum of the tensile stresses equals the sum of the compressive stresses.
8. The moment of the stresses in any section must equal the moment of the applied loads on either side of the section.

The maximum stress in the concrete is about 15 per cent less when calculated by the parabolic theory proposed by Professor Talbot than by the straight line theory given below.

Notation.*—The following notation will be used:

b = breadth of rectangular beam.

d = distance from the compression face to the center of the metal reinforcement.

d' = distance from the center of the reinforcement to center of gravity of compressive stresses.

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* Notation adopted by Professor Talbot in University of Illinois Engineering Experiment Station Bulletin, No. 4.

k = ratio of distance between compression face and neutral axis to distance d.

z = distance from compression face to center of gravity of compressive stresses.

A = area of cross-section of metal reinforcement.

$p = \frac{A}{bd}$ = ratio of area of metal reinforcement to area of concrete above center of reinforcement.

o = circumference or periphery of one reinforcing bar.

m = number of reinforcing bars.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete in compression.

$n = \frac{E_s}{E_c}$ = ratio of two moduli.

f = tensile stress per unit of area in metal reinforcement.

c = compressive stress per unit of area in most remote fiber of concrete.

ϵ_s = deformation per unit of length in the metal reinforcement.

ϵ_c = deformation per unit of length in most remote fiber of the concrete.

M = bending moment at the given section.

s = horizontal tensile stress per unit of area in the concrete.

t = diagonal tensile stress per unit of area in the concrete.

u = bond stress per unit of area on the surface of the reinforcing bars.

v = vertical shearing stress and horizontal shearing stress per unit of area in the concrete.

Distribution of Stresses in Beams.—In (c), Fig. 29, b is the breadth, d is the depth of the beam above the center of the steel, and $k.d$ is the distance of the neutral axis below the top of the beam, k being a ratio.

In (a) the deformations are shown to be proportional to the distance from the neutral axis, and in (b) the stress in the steel is n times the stress in concrete at the same distance from the neutral axis.

Now in (b), Fig. 29, we have summation of horizontal compressive stresses equal to horizontal tensile stress in steel, and

$$\tfrac{1}{2}c.k.d = \frac{f.A}{b} \tag{51}$$

$$= f.p.d$$

$$\text{since } p = \frac{A}{b.d}, \text{ and}$$

$$\tfrac{1}{2}c.k = f.p \tag{52}$$

We also have from (b) that

$$c : \frac{f}{n} :: k.d : d(1 - k)$$

$$f.k.d = c.n.d(1 - k),$$

$$fk = cn(1 - k) \tag{52a}$$

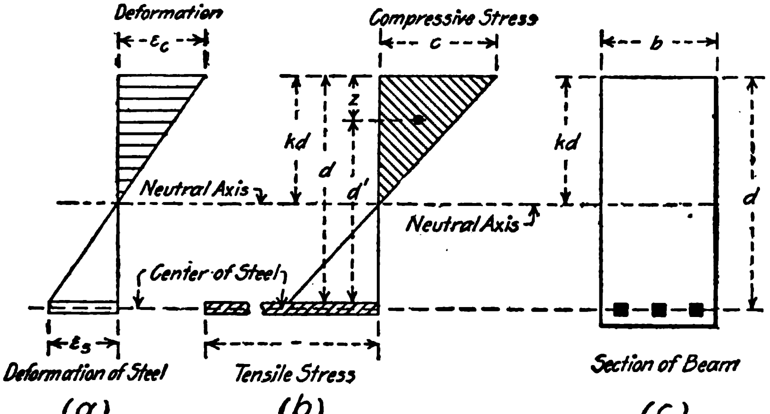

FIG. 29.

And substituting the value of f from (52)

$$\tfrac{1}{2}c.k^2 = c.p.n(1 - k)$$

$$\tfrac{1}{2}k^2 = p.n(1 - k)$$

$$k = -p.n + \sqrt{2p.n + p^2.n^2} \tag{53}$$

Since the bending moment equals the resisting moment, if M = the bending moment per inch of width, then

$$M = \tfrac{1}{2}c.k.d.d' = \frac{A.f.d'}{b} \tag{54}$$

$$= \tfrac{1}{2}c.k.d^2(1 - \tfrac{1}{3}k) = \frac{A.f.d}{b}(1 - \tfrac{1}{3}k) \tag{55}$$

Hence the formula for the strength of a rectangular beam per inch of width in terms of strength of steel, f, is

$$M = \frac{A.f.d}{b}(1 - \tfrac{1}{3}k) \tag{56}$$

And in terms of compressive strength of concrete, c,

$$M = \tfrac{1}{2}c.k.d^2(1 - \tfrac{1}{3}k) \tag{57}$$

In Fig. 30 values of k have been plotted for values of p from 0.002 (0.2 per cent) to 0.024 (2.4 per cent), and for $n = 10$, $n = 15$, and $n = 20$.

The allowable bending moments for $f = 10{,}000$ lbs. per sq. in.* for different values of p and n are plotted for concrete beams 12 inches wide and from 4 to 36 inches deep, varying by 2 inches in depth. Corresponding values of the compression in the concrete are given on the right of the diagram. The use of the diagram will be illustrated by solving two problems:

Problem 1.—Required the safe uniform load including the weight of the beam for a reinforced concrete beam 12 inches wide and 24 inches deep to the center of reinforcement, and 20 feet long, for $f = 10{,}000$ lbs. per sq. in., $p = 0.01$ (1 per cent), concrete 1–3–6.

Solution.—For 1–3–6 concrete $E_c = 2{,}000{,}000$ and $n = 15$. Entering the right hand table on the lower edge with $p = 0.01$ pass up to the curve $n = 15$, and on the right edge the stress in concrete, c, equals 480 lbs., and on the left edge $k = 0.42$ and $(1 - \tfrac{1}{3}k) = 0.86$. Then on horizontal line $(1 - \tfrac{1}{3}k) = 0.86$ pass to the left until it intersects the inclined line " Beam 24″," then pass down on the vertical line passing through the intersection of the horizontal line and the " Beam " line until the inclined line $p = 0.01$ is cut; then on the horizontal line

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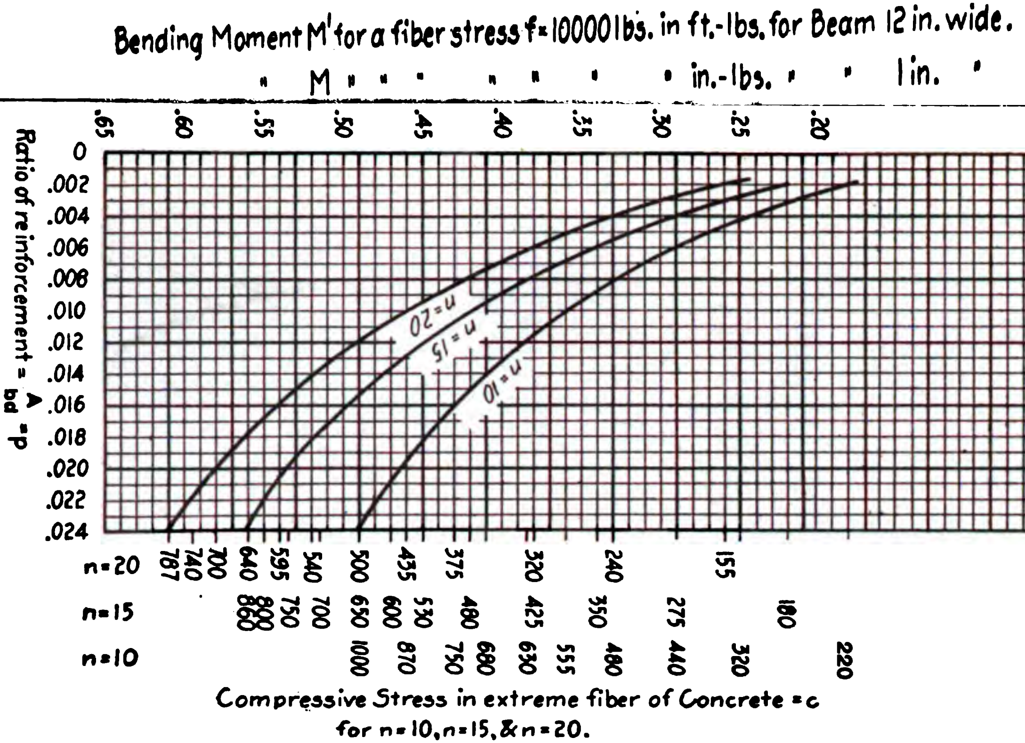
* Factor of safety of 6 in the steel.


Bending Moment M' for a fiber stress f=10000 lbs. in ft.-lbs. for Beam 12 in. wide.
" M " " " " " " " in.-lbs. " " 1 in. "
.65
.60
.55
.50
.45
.40
.35
.30
.25
.20
Ratio of reinforcement = A/bd = p
0
.002
.004
.006
.008
.010
.012
.014
.016
.018
.020
.022
.024
n=20
n=15
n=10
n=20
787
740
700
640
595
540
500
435
375
320
240
155
n=15
860
800
750
700
650
600
530
480
425
350
275
180
n=10
1000
870
750
680
630
555
480
440
320
220
Compressive Stress in extreme fiber of Concrete = c
for n=10, n=15, & n=20.

passing through the intersection of the vertical line and the line of $p = 0.01$ pass to the left edge where we find

$$M' = 50{,}000 \text{ ft.-lbs.}$$

To find the safe uniform load we have

$$M' = \tfrac{1}{8}W.L, \text{ and}$$

$$W = 20{,}000 \text{ lbs., or}$$

$$w = 1{,}000 \text{ lbs. per lineal foot.}$$

The area of steel reinforcement, A, is given by the intersection of the line $p = 0.01$ and " Beam " line and is a little less than 3 sq. in.

The entire depth of beam would be 26″, and the safe load per lineal foot exclusive of weight of beam would be $w' = w - 325 = 675$ lbs.

If p were taken equal to 0.02 we would have $c = 750$ lbs.; $k = 0.535$; $M' = 95{,}000$ ft.-lbs.; $w = 1900$ lbs.; and $w' = 1575$ lbs. Which gives a much greater safe load but a much larger stress in the concrete than is usually allowed.

For a fiber stress in the steel $f = 15{,}000$ lbs. per sq. in. in the problem above, add 50 per cent to the stress in the concrete on the right and 50 per cent to the allowable bending moment on the left.

Problem 2.—Required the depth of a reinforced concrete beam 12 inches wide that will carry a bending moment of 80,000 ft.-lbs., $f = 10{,}000$ lbs. per sq. in.; $p = 0.02$ (2 per cent), using 1–2–4 concrete.

Solution.—For 1–2–4 concrete $E_c = 3{,}000{,}000$ and $n = 10$. Now enter the table on the right with $p = 0.02$ and pass up a vertical line to curved line $n = 10$; then on the right $c = 870$ lbs.; and on the left $k = 0.462$. Following across the second diagram to the left as in Problem 1, we see that a beam 22 inches deep will carry 82,000 ft.-lbs., which is sufficient.

This beam is designed for a factor of safety of about 4 for the concrete and about 6 for the steel. The reinforcement should be $p = 0.01$, for which a 30″ beam is required. In this beam $c = 555$ lbs.

Ratio of Reinforcement and Working Stresses.—From the diagram in Fig. 30 it will be seen that to obtain the same factor of safety in both concrete and steel the reinforcement should be as follows: For 1–2–4 concrete, $p = 0.01$ (1 per cent); for 1–3–6 concrete, $p = 0.009$ (0.9 per cent); for 1–4–8 concrete, $p = 0.007$ (0.7 per cent). Excessive reinforcement results either in a waste of steel or in excessively high compressive stresses in the concrete.

Bond or Resistance to Slipping of Reinforcing Bars.—Where there is no web reinforcement the shear is taken by the concrete and the shear increments are transferred to the bars by the adhesion of the concrete to the bars. The solution is the same as that for finding the pitch of rivets in the flanges of a plate girder.

Now in (b), Fig. 29, take two right sections at a distance dx apart. Equilibrium of these two sections is maintained by the resisting moment of the bond which is equal and opposite to the moment of the vertical shear, a couple with an arm dx.

Taking moments about center of gravity of compressive forces we have

$$V.dx = m.o.u.dx.d' \qquad (58)$$

where $m =$ number of bars, $o =$ surface of bar for one inch in length, $u =$ bond developed per square inch of surface of bar, and V is the vertical shear in the beam.

Solving for u, we have

$$u = \frac{V}{m.o.d'} \qquad (59)$$

Equation (59) applies to the case of horizontal bars. For inclined bars d' will be a variable and u will be the horizontal component of the bond resistance.

Vertical and Horizontal Shearing Stresses.—At any point in a beam the vertical unit shearing stress is equal to the horizontal unit shearing stress. The horizontal shearing stress transmits the increments of tension to the reinforcing bars by bond stresses, as explained in the preceding discussion.

The amount of this horizontal stress transmitted to the reinforcing bars is by equation (59)

$$m.o.u=\frac{V}{d'}$$

Now if the horizontal shear just above the plane of the bars is v, the total horizontal shearing stress will be $v.b$, and

$$v=\frac{V}{b.d'} \tag{60}$$

As no tension is assumed to exist in the concrete, the horizontal shear will be constant up to the neutral axis, above which point it decreases to zero at the top of the beam. It will be seen that lean or poor concrete lacking in shearing strength should not be placed below the neutral axis of beams with the idea that it may be satisfactory for the reason that the concrete is assumed to take no tension.

Diagonal Tension in Concrete.—In Mechanics of Materials (Merriman's Mechanics of Materials, p. 265, 1905 edition) it is shown that shear and tensile stresses combine to cause diagonal tensile stresses.

$$t=\tfrac{1}{2}s+\sqrt{\tfrac{1}{4}s^2+v^2} \tag{61}$$

Where t is the diagonal tensile unit stress, s is the horizontal tensile unit stress, and v is the horizontal or vertical shearing unit stress. The direction that stress t makes with the horizontal is one-half the angle whose cotangent is $\tfrac{1}{2}s/v$. If there is no tension in the concrete this reduces to

$$t=v \tag{62}$$

and t makes an angle of 45° with the horizontal.

Stresses due to diagonal tension may be carried (1) by bending the reinforced bars, or strips sheared from them, into a diagonal position, or (2) by means of stirrups to take the vertical component of the diagonal tension.

Depth of Concrete Below Bars.—For adequate fire protection a thickness of 2 inches is necessary. Where fire protection is not required Edwin Thacher uses a thickness of 1″ for beams 4 inches deep, 2″ for beams 20 inches and over, and proportional thickness for depths between 4 and 20 inches.

Spacing of Bars.—An arbitrary rule is to space bars no nearer together in the clear than two diameters, and in no case less than 1½ inches apart, nor nearer than 1½ inches to either side of the beam.

Length of Bar to Imbed in Concrete.—Experiments show that plain bars should be imbedded for a depth of not less than 80 diameters, and deformed bars for not less than 50 diameters. Where bars can not be imbedded for this length they should be bent or anchored.

Expansion and Contraction.—The coefficient of expansion of concrete is practically the same as for steel, about 0.0000065. Reinforced retaining walls are commonly built without expansion and contraction joints. To prevent cracks reinforcement is placed in the direction of the length of the wall.

Mr. A. L. Johnson gives the following formula for the amount of reinforcing required to prevent temperature cracks:*

$$\text{Area of steel} = \frac{\text{Tensile strength of concrete}}{\text{Elastic limit of steel}} \times \text{area of concrete.}$$

For mild steel the elastic limit is about 33,000 lbs. per sq. in.; tensile strength of concrete is about 200 lbs. per sq. in.; and area of steel $= \frac{1}{165}$ of the area of the wall. For high steel the elastic limit is about 55,000 lbs. per sq. in., and the area of the steel required $= \frac{1}{275}$ of the area of the wall.

Mr. W. W. Colpitts recommends 0.6 sq. in. of steel per sq. ft. of concrete.†

To provide for expansion, contraction and shrinkage 0.6 per cent of steel would appear ample.

High or Mild Steel.—With ordinary unit stresses in concrete and steel, little is to be gained from the use of steel with a high elastic limit. Experiments and practice would appear to show that as good results may be obtained, for ordinary purposes, by the use of plain bars, as by the use of any of the various deformed bars.

The ultimate strength of mild steel may be taken as 60,000 lbs. per sq. in. For retaining walls subject to shock use a factor of safety of 6, while for quiescent loads use a factor of safety of 4.

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* Railroad Gazette, March 13, 1903.

† Railway Age, January, 1904.

The above discussion of reinforced concrete is very brief but is sufficient for the design of retaining walls.

DESIGN OF REINFORCED CONCRETE RETAINING WALLS.—The design of reinforced concrete retaining walls will be illustrated by means of two examples.

Problem 1.—Design a reinforced concrete retaining wall of the beam type to carry a sand filling which is 16 ft. high, weighs 100 lbs. per cu. ft., and has an angle of repose, $\phi = 35°$, and slopes back at the

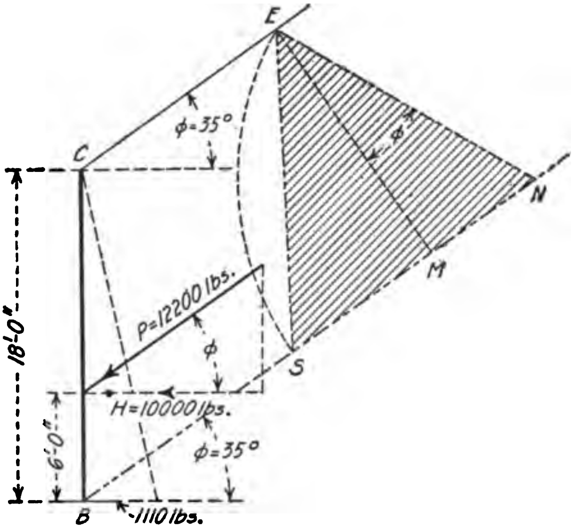

FIG. 31.

angle of repose. Concrete to be 1–3–6 Portland cement, and factor of safety of concrete and steel to be 6.

Solution. Vertical Beam.—The bottom of the foundation will need to be about 4 feet deep and we will assume the stem of the wall to be 18 feet high. In Fig. 31 the pressure $P = \Delta SEN.w = 12{,}200$ lbs., and is parallel to the top surface. The horizontal component of P is $H = 10{,}000$ lbs. The bending moment about B is $M' = 10{,}000 \times 6 = 60{,}000$ ft.-lbs.

Now in using the diagram, Fig. 30, take $n = 15$ and $p = 0.01$. Then we find that for a beam 28″ deep $M' = 67{,}000$ ft.-lbs. Adding 2 inches for protection of the steel, the wall will be taken as 30 inches

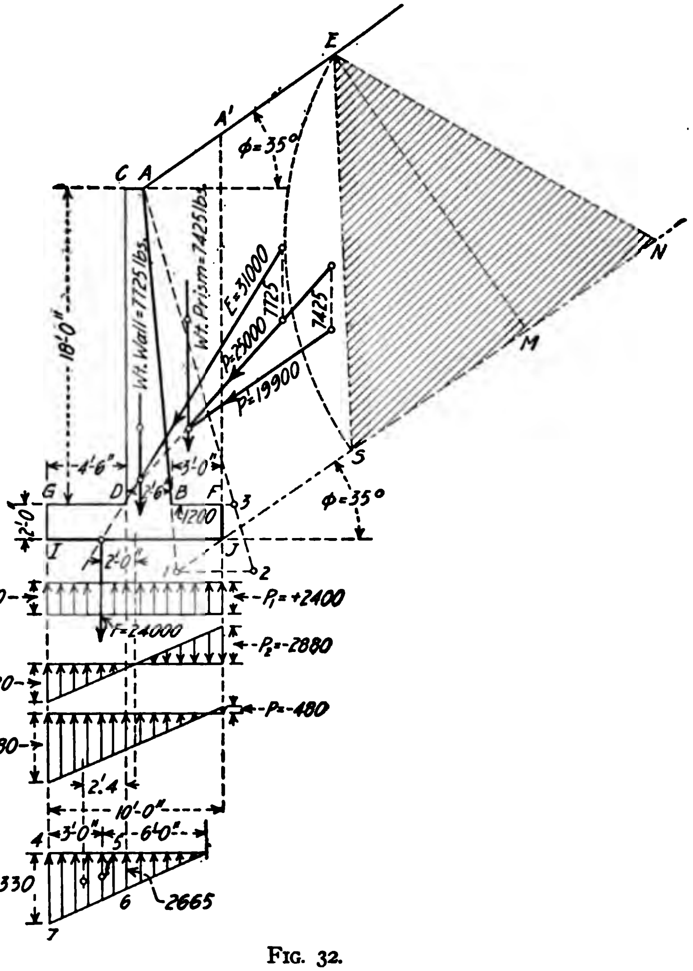

FIG. 32.

thick. The top will be made 12 inches thick. To carry this moment 3 square inches of steel will be required, and bars 1″ ☐ will be spaced 4 inches center to center.

Foundation.—We will assume that the footing is 2 feet thick and 10 feet long, as shown in Fig. 32. Then in Fig. 32 the pressure on plane

A'-F is $P' = 19{,}900$ lbs., the weight of the earth prism A-A'-B-F is 7,425 lbs., and $P = 25{,}000$ lbs. Combining P and the weight of the wall $= 7{,}725$ lbs., we have $E = 31{,}000$ lbs., which cuts the base 2 feet to the left of the center, outside of the middle third.

Now $F = 24{,}000$ lbs. and $p_1 = +2{,}400$ lbs. per sp. ft., $p_2 = \frac{6F.b}{d^2} = +2{,}880$ lbs. per sq. ft., and $p = +5{,}280$ and -480 lbs. per sq. ft. Now since the foundation cannot take tension, we will have from equation (50) $p' = \frac{2}{3}F/a$ where $a = 3$ feet, and $p' = 5{,}333$ lbs. per sq. ft.

This pressure is safe for a good gravel, or sand and clay. While the resultant cuts outside the middle third, the base is sufficiently long for the conditions assumed. To calculate the bending moment to the left of D take the area of the lower stress diagram 4–5–6–7 and multiply it by the distance of its center of gravity to the left of D, then

$$M' = \left(\frac{5{,}333 + 2{,}665}{2}\right) 4\tfrac{1}{2} \times 2.4' = 43{,}200 \text{ ft.-lbs.}$$

Now with $n = 15$, and $p = 0.01$, we find in the diagram that for a beam 22 inches deep $M' = 42{,}000$ ft.-lbs. This is very close and the foundation of 24 inches assumed will be used. Steel bars 1″ ☐ will be spaced 4 inches center to center the full length of the foundation. Bars

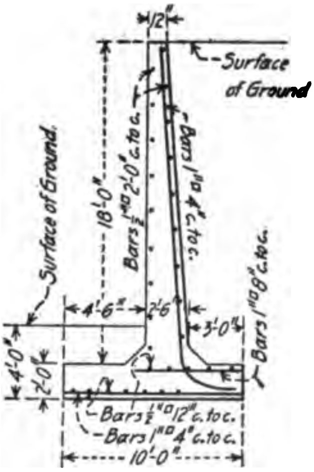

FIG. 33.

1″ □ and spaced 8 inches center to center will be placed 2 inches from the top in the inner toe.

To prevent longitudinal cracks, horizontal bars will be spaced as shown. This reinforcement is somewhat less than would be required by Johnson's formula, but agrees well with practice.

Bond.—The tendency for the bars to slip at the bottom of the upright beam is

$$u=\frac{V}{m.o.d'} \tag{59}$$

Now $V=10{,}000$ lbs., $m=3$, $o=4$ sq. in., $d'=24$ inches, and $u=35$ lbs. per square inch, which is entirely safe.

Vertical Shear.—At the point D as above we have

$$v=\frac{V}{b.d'}=35 \text{ lbs. per sq. in.} \tag{60}$$

If smooth bars are used in the vertical beam they should be bent and anchored at the bottom. For detailed specifications for the concrete see Am. Ry. Eng. and M. of W. Assoc. specifications in Appendix I.

Problem 2.—Design a reinforced concrete retaining wall with counterforts to carry a sand filling 17½ feet above the ground, which weighs 100 lbs. per cu. ft., has an angle of repose, $\phi=37°\ 30'$, and carries a railroad track which is equivalent to a surcharge of 6 feet. Concrete to be 1–3–6 Portland cement, and the factor of safety of concrete and steel to be 4. Counterforts are to be spaced 10 feet center to center, as shown in Fig. 35.

Solution.—The pressure P' on the vertical plane 2–B is calculated graphically, as shown in Fig. 34. $\triangle\ SeN\,.\,w$ equals the pressure on the vertical plane B–6, and the pressure triangle is B–6–4. Resultant pressure, P', acts through the center of gravity and is equal to the area B–2–3–4, equals 9,200 lbs. Resultant pressure, P'', acting on plane G–2, is found to be 7,700 lbs.

Now the weight of the prism of filling O–I–2–G is 15,000 lbs., and combining this weight with P', we have $P=17{,}500$ lbs., acting as shown. The weight of the concrete wall per lineal foot is approximately 6,500 lbs., which, when combined with P, gives $E = 23{,}200$ lbs. Resultant E cuts the base at a distance 1.6 feet from the center, and the vertical component of E is $F = 21{,}500$ lbs.

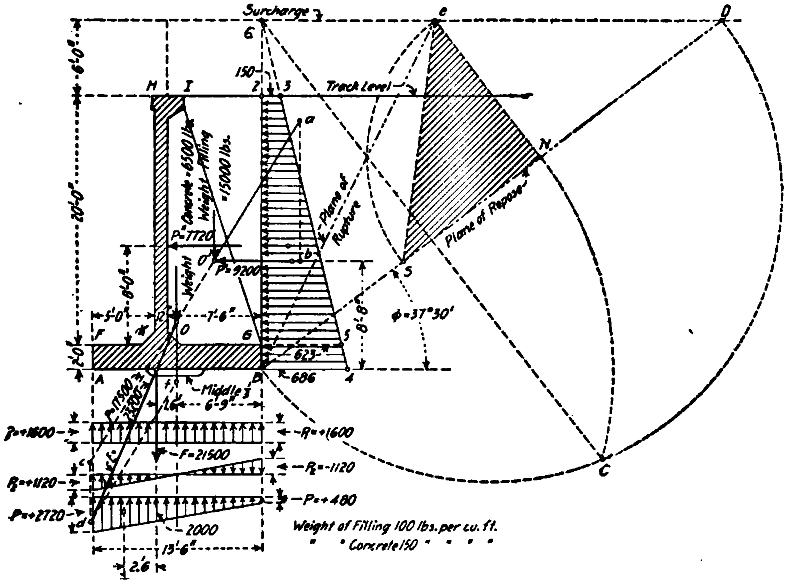

FIG. 34.

Vertical Wall.—In designing the center slab the span will be taken as 10′ 0″ (where the wall has no cracks the actual span is less than the clear span of 8′ 6″). Now taking the bottom strip one foot wide and 10′ 0″ long, we are to design a simple beam that will carry a load of 623 lbs. per lineal foot, or $M' = \frac{1}{8} \times 623 \times 10^2 = 7{,}788$ ft.-lbs. For $n = 15$, and $p = 0.01$, we find in diagram Fig. 30, that a beam 10 inches deep will carry 8,000 ft.-lbs., for $f = 10{,}000$ lbs.; and 12,000 ft.-lbs., for $f = 15{,}000$ lbs. per sq. in., which is more than ample but will be used. The thickness of the slab will then be 12 inches. This slab will require 1.2 sq. in. of steel per foot of height. Bars ¾″ □ and ½″ □ will be used and will be spaced as shown in Fig. 35. The centers of the bars will be spaced 2″ from the face of the wall. To take up stresses due to unequal settlement bars ½″ □ will be placed vertical, as shown in Fig. 35.

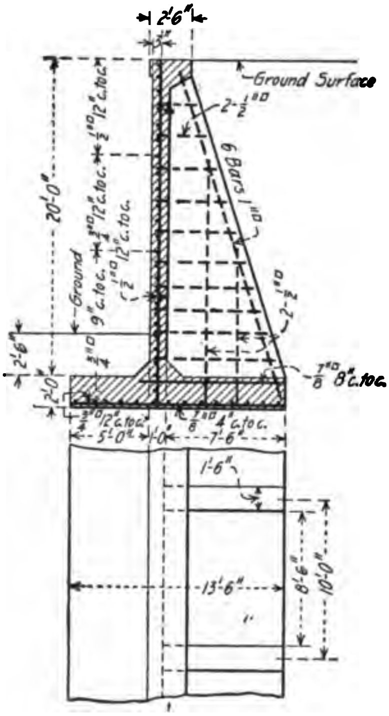

FIG. 35.

Counterforts.—The bending moment on a counterfort at O-G in Fig. 34 will be $M' = 7{,}720 \times 8 \times 10 = 617{,}600$ ft.-lbs. The counterforts will be taken as 18 inches wide. With $n = 15$, and $p = 0.01$, we have for a beam 8 feet deep and 18″ wide

$$M = A.f.d' = A.f.d(1 - \tfrac{1}{3}k) \qquad (56)$$

From diagram, Fig. 30, $(1 - \frac{1}{3}k) = 0.86$,

$$A = 17.3 \text{ sq. in., and}$$

$$M = 15{,}000 \times 17.3 \times 6.9$$
$$= 1{,}790{,}000 \text{ ft.-lbs.}$$

This shows that too much steel has been used. It would also be impossible to use 17.3 square inches of steel in an 18-inch counterfort.

Try $p = 0.004$, then $(1 - \frac{1}{3}k)$ equals 0.903, and

$$M = A.f.d(1 - \tfrac{1}{3}k)$$

$$A = 6.9 \text{ sq. in., and}$$

$$M = 15{,}000 \times 7.2 \times 6.9$$

$$= 745{,}200 \text{ ft.-lbs.}$$

This shows that the reinforcement is still somewhat large. Six bars 1″ □ will be used, and will be placed 3 inches from the outside of the concrete. The bars should be bent back under the footing and anchored to prevent slipping. Bars ½″ □ will be placed in the counterforts as shown, to take the horizontal and vertical shear.

Foundation.—In Fig. 34 the direct pressure $p_1 = +1{,}600$ lbs. per sq. ft., while the pressure due to moment is

$$p_2 = \frac{6F.b}{d^2} = \pm 1{,}120 \text{ lbs. per sq. ft.}$$

Then $p = +2{,}720$ and $+480$ lbs. per sq. ft., which is entirely safe for ordinary conditions.

The maximum moment at K in the outer toe is found in Fig. 34 by taking the moment of the moment area to the left of K, and is

$$M' = \left(\frac{2720 + 2000}{2}\right) 5 \times 2.6$$

$$= 30{,}680 \text{ ft.-lbs.}$$

From diagram Fig. 30, with $n = 15$, $p = 0.01$, and $f = 15{,}000$ lbs. per sq. in., a beam 16 inches deep is sufficient. This would make the total depth of slab 18 inches, and the steel reinforcement would be 2 sq. in. per lineal foot. However, a slab 24 inches deep has been assumed and bars ⅞″ □ spaced 4 inches center to center will be used near the bottom and ⅞″ □ rods spaced 8 inches center to center near the top of the slab as shown.

Longitudinal bars ¾″ □ spaced 12 inches center to center will be used near the bottom as shown.

General Principles of Design.—The same precautions should be observed in designing reinforced concrete retaining walls as in the design of masonry retaining walls—which see. Details of placing the forms and the concrete of a number of important retaining walls are given in Chapter VII. Care should be used in placing the steel in the forms and in depositing the concrete in the forms. The concrete should be well mixed and should be deposited very wet in order to properly coat the reinforcing bars and to make a uniformly dense concrete. If reinforced concrete is properly made the thin layer of grout that clings to the steel acts as a perfect protection against corrosion. For detailed specifications for concrete and cement see Appendix I, also see Chapter V.

CHAPTER V.

EXPERIMENTS ON RETAINING WALLS.

Experiments.—The preceding investigation has been based on the ideal condition of a granular mass and recourse must be had to experiments to check up the theoretical deductions. Experiments on earth pressures have been made by a large number of engineers, and a brief summary of the results will be given.

Experiments by M. M. L. Leygue.—These experiments which are described in detail in Cain's "Retaining Walls," were first published in "Annales des Ponts et Chaussees" in 1885. The experiments were made with dry sand, using a small model 1.22 to 1.3 feet long and 0.66 to 0.82 feet high, and appear to prove the following:

Cohesion.—That cohesion exists in a mass of sand but is very small and may be neglected.

Surface of Rupture.—All particles lying between the back of the wall and a convex surface (surface of rupture) moved parallel to the surface of rupture. The surface of rupture roughly bisected the angle between the back of the wall and the angle of repose of the sand; thus differing from theory for high surcharge.

Friction on the Wall.—The full friction was exerted on the back of the wall by the sand.

Ratio of Horizontal to Vertical Pressure.—Leygue found that the ratio of the horizontal to vertical pressure (k) for a vertical wall, varied from .30 for a horizontal surcharge to .65 for a surcharge sloping at the angle of repose of 33° 42′.

Center of Pressure.—Leygue obtained values of the distance of the point of application of the resultant pressure above the base varying from $.296h$ to $.375h$, with an average of $.34h$.

Experiments by Sir Benjamin Baker.—These experiments were published in the Minutes of Proc. Inst. of C. E., Vol. LXV, p. 140.

The experiments showed that the coarser the materials the less lateral pressure. The lateral pressure for coal, shingle, ballast, and macadam material was about one-tenth the vertical pressure, and varied uniformly with it. The lateral pressure in clay was about one-fifth of the vertical, while in loose earth it varied from one-tenth to one-fifth.

Experiments by E. P. Goodrich.—In Trans. Am. Soc. C. E., Vol. LIII, pp. 272 to 304, Mr. Goodrich has recorded certain experiments made by him and has also discussed the principal experiments made by other experimenters. In his experiments he used the following apparatus:

1. A wooden model 3 ft. × 3 ft. and 6 ft. deep. The pressure of the moist sand was measured by means of a lever apparatus and a scale beam. The ratio of the horizontal to vertical pressure was about one-fifteenth plus 15 lbs. The arch action of the sand made the results of little value as applied to retaining walls.

2. An experiment was tried on sheet piling driven to form the face of a retaining wall. A box 6 ft. × 1 ft. × 15 ft., inside measurement, was built with one side open, which side was placed against the sheet piling and the other broad side received the pressure of the filling. The side which received the earth pressure was made of 3 × 10-in. yellow pine plank, 15 feet long. The deflection of this side was measured at intervals for three months when final settlement appeared to have taken place. The actual pressures of the earth on the wall were obtained by loading a similar plank with bricks. The results were also checked by comparing the observed deflections of the wall with calculated deflections. The material in the fill was fine beach sand weighing about 100 lbs. per cu. ft. These experiments showed that the horizontal pressure was about one-half the vertical pressure.

3. Experiments were made on the testing machine shown in Fig. 36. A cast iron cylinder was bored out to a diameter of 6 ins. and a depth of 5 ins. A plug was fitted in it and was attached to the head of an ordinary testing machine. A 1-in. hole was bored in one side of the cylinder near the bottom, and a plug was carefully fitted to the hole. The plug was just long enough to reach through the wall of the cylin-

der. The faced portion of the outside of the cylinder was fitted with hard rubber insulation opposite the 1-in. hole. On these hard rubber pieces were fitted pieces of copper which were connected to an electric battery with an ordinary buzzer in the circuit. When the plug was pushed home the underside of the plug touched the copper and completed the circuit. If the plug was pushed slightly outward the circuit was broken and the buzzer stopped. Experiment showed that a movement of 0.001 inch was sufficient to break the circuit while a movement of 0.0002 was indicated by the tone of the buzzer. The pressure was measured by means of the steel-yard as shown.

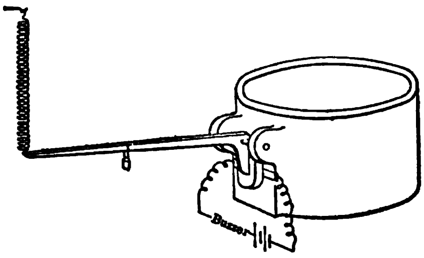

FIG. 36.

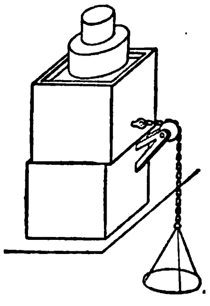

FIG. 37.

In making experiments the pressure was applied until the circuit was broken, when the weights on the steel-yard were moved out until the circuit was closed, the vertical pressure was then applied until the circuit was broken, weights were moved out on the steel-yard until the circuit was closed, and so on until the experiment was completed. The pressure was relieved in the reverse manner and the mean taken as the true values.

Internal Friction.—The angle of internal friction was obtained by means of the apparatus shown in Fig. 37. The two boxes are one foot in each internal dimension, but without tops, the lower one only having a bottom. In conducting an experiment the two boxes were filled with the earth to be tested and enough weights were added to the scale pan to start the upper box to sliding. It was then checked and jerked back into position, known weights were added to the top and weights were added to the scale pan until sliding took place. The angle of internal friction was determined from Rankine's formula $\sin\phi = \frac{1-r}{1+r}$.

The angle of surface slope was found by measuring the slope that the material assumed when loosely piled.*

The angles of internal friction, angles of surface slope and ratio of lateral to vertical pressure for various materials as determined by Goodrich and others are given in Table IX.

TABLE IX.

ANGLES OF INTERNAL FRICTION AND OF SURFACE SLOPE FOR DIFFERENT MATERIALS.

Ratio of Lateral to Vertical Pressure.	Kind of Material.	Observer.	Tangent of Angle of Internal Friction.	Tangent of Angle of Surface.	Observer.
Percentage.					
10..........	Coal, shingle, ballast, etc.	Baker.	1.423	1.11–0.70	Rankine.
	Bank sand.	Goodrich.	1.423	1.45–0.60 1.00–0.67 0.75–0.38	Goodrich. Trautwine. Rankine.
15..........	Rip-rap.	Goodrich.	1.097	1.00	Goodrich.
	Earth.	Baker.	1.097	0.66	Trautwine.
20..........	100-up quicksand.	Goodrich.	0.895		
	Clay.	Baker.	0.895	1.00	Rankine.
25..........	50-100 quicksand.	Goodrich.	0.750		
	Earth.	Steel.	0.750	0.66 0.58	Trautwine Steel.
	Bank sand.	Wilson.	0.750	0.58–0.62	Wilson.
35..........	50-100 sand.	Goodrich.	0.549	0.85	Goodrich.
	Bank sand.	"	0.549	0.75–0.38	Rankine.
40..........	Clay.	"	0.474	1.00	Rankine.
	Cinders.	"	0.474	0.86	Goodrich.
	½ inch gravel.	"	0.474	0.66	Trautwine
50..........	¼ inch gravel.	"	0.350	0.85	Goodrich.
	Bank sand.	"	0.350	0.75–0.38	Rankine.
60..........	30-50 sand.	"	0.258	0.66	Goodrich.
70..........	20-30 sand.	"	0.179	0.75–0.38	Rankine.

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* The angles of internal friction should be used in retaining wall calculations.

Compressibility of Soils.—Tests made on the compressibility of soils showed, (1) that layers immediately beneath the compressing disk were compressed more than the more distant ones, and that a compressed cone formed under the disk; (2) that an appreciable compression took place only with great loads; (3) that under extreme pressures garden earth and sand showed quite an elastic reaction; and (4) that clay, even when containing a small amount of water, would "crawl" and relieve the pressure by squeezing through openings in threads or sheets.

Results.—Goodrich discussed the foregoing experiments and experiments by Sir Benjamin Baker; G. H. Darwin,* A. A. Steel,† and George Wilson,‡ and arrived at the following conclusions:

Saturation.—The lateral pressure in materials which differ only in the percentage of saturation, varies as the ordinates to a curve of the fourth degree, possessing two minimums and one maximum between 0 and 100 per cent of saturation. The effect of moisture is more marked at heavy vertical pressure than at lesser ones.

Neither saturated nor loose dry materials are apt to exert the greatest lateral thrust, and, with slight saturation and rather moist conditions, the lateral thrust is apt to be relatively small.

In moist earths the first large application of pressure is likely to produce a permanent set which exerts lateral pressure at low repetitions of pressures.

Rankine's theory of conjugate pressures is correct when the proper angle of friction is found, and probably adaptations of his formulas will be of most practical value.

Conclusion.—From the preceding experiments it would appear that 'Rankine's Theory' furnishes the best solution yet proposed for the calculation of the pressure on retaining walls.

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* Minutes of Proceedings, Inst. C. E., Vol. LXXI, p. 350.

† Engineering News, October 19, 1889, Vol. XLII, p. 261.

‡ Minutes of Proceedings, Inst. C. E., Vol. CXLIX, p. 208 (271).

CHAPTER VI.

EXAMPLES OF RETAINING WALLS.

Concrete Retaining Wall, Illinois Central R. R., Chicago, Ill.[*]—The concrete retaining wall shown in Fig. 38 was built by the Illinois Central R. R., on the lake front in Chicago in 1905. The wall is carried on piles 30 feet long, spaced 3 feet center to center parallel to the wall, and 3 ft. 4⅜ ins. transversely. The wall is designed for an 8-ft. surcharge, with this surcharge the resultant load is 31,750 lbs. per linear foot of wall, or 15.87 tons per pile.

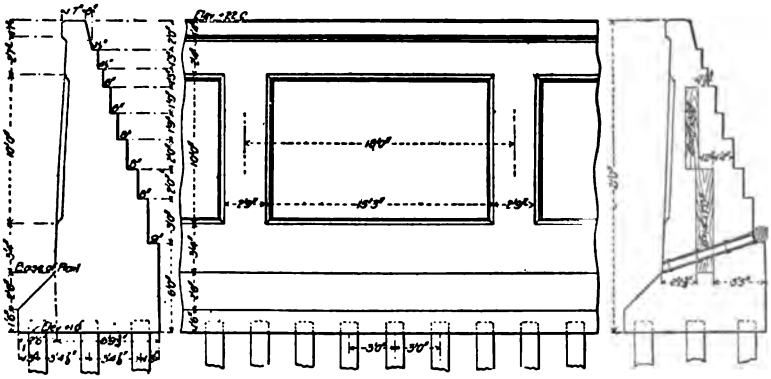

FIG. 38.—CONCRETE RETAINING WALL, ILLINOIS CENTRAL R. R., CHICAGO, ILL.

The concrete was mixed wet in the proportions 1 part Portland cement, 3 parts sand, and 6 parts broken limestone. The front face of the wall and the coping were finished with mortar as the concrete was laid, the remainder of the concrete was left rough.

The concrete was laid in four courses. The first course extends from the bottom up to the base of the rail, this course being laid without joints, continuously as possible. Between that level and the bottom of the panel is the second course, in which expansion joints are placed every 108 feet on the center line of the pilasters between the panels. The third course extends from the bottom of the panel to the bottom of the coping and contains expansion joints every 54 feet. The coping makes the fourth course and was laid before the third one was set. Each of the first three courses was sloped 2 inches on top and had a 6 × 12-in. timber laid in the top to make a key that would bond the adjacent faces of the courses together. A 6-in. drain tile was placed longitudinally along the back of the wall on the tread of the upper 9-in. step and connected at intervals with a pipe through the wall. The expansion joints were filled with one thickness of Hydrex felt, and the concrete was laid in alternate sections so that there are true parting planes at the expansion joints. The end of the section laid first was bulkheaded and vertical timbers were nailed to the inside of the bulkhead to form a key way for the end of the next section.

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[*] Engineering Record, January 27, 1906.

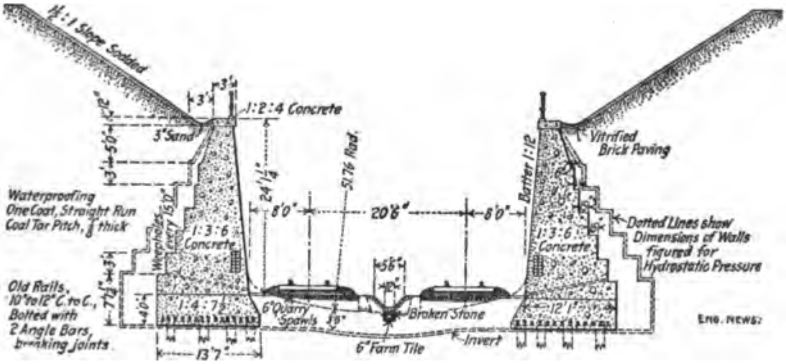

FIG. 39.—CONCRETE RETAINING WALLS FOR DETROIT TUNNEL.

Retaining Walls for Detroit Tunnel.*—The retaining walls shown in Fig. 39 were designed for the open cut approach of the proposed tunnel (1906) under the Detroit River for the Michigan Central R. R. It will be seen that the walls are braced apart by a concrete invert. The walls are made of concrete with the proportions 1 part Portland cement, 3 parts sand, and 6 parts broken stone, for the body and

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* Engineering News, February 15, 1906.

1-4-7½ for the footing course. The footings are reinforced with old rails, 10 to 12 in. centers; the main body of the wall is not reinforced. The back of the walls are coated with a layer of tar ⅛-in. thick. Weep holes are provided through the wall every 15 feet. Transverse tar paper joints are made every 50 feet to provide for expansion.

Retaining Walls on the Pennsylvania Ave. Subway, Philadelphia, Pa.*—Fig. 40 shows a typical section of retaining wall with standard coping and French drain and weeper. Reduced heights were usually designed by cutting the standard section at the desired point. Wherever possible, the faces of all walls were given a batter of ½ in. per foot. The neat line was generally 18 ins. below the top of the rail. Unless otherwise ordered, the bed of the foundation was specified to be 5 ft. below the top of the rail in earth and soft rock, and 3 ft. below in hard rock. On lump-sum contracts, any masonry below these lines was paid for as additional masonry at the prices bid.

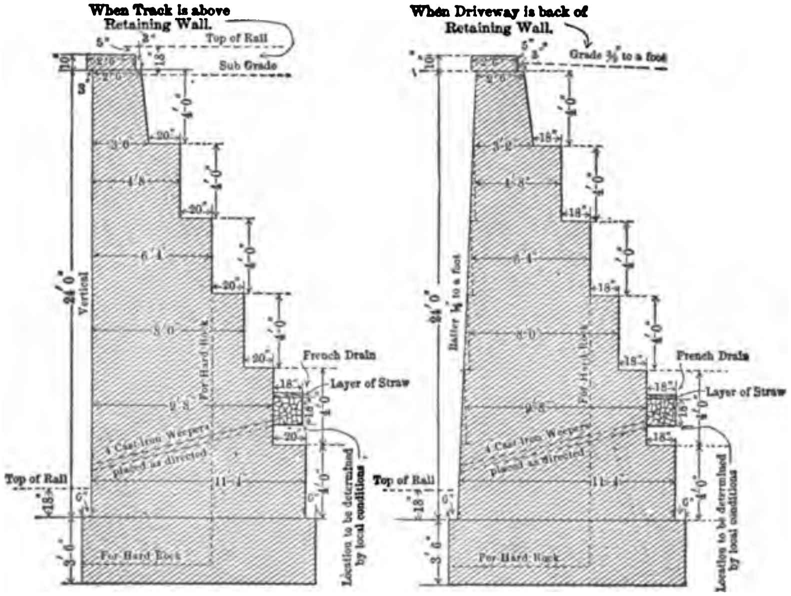

FIG. 40.

Specifications.—Masonry.—" All retaining walls shall consist of first-class Conshohocken or other approved stone of good shape and good flat beds; no stone having less bed than face; laid on their broadest faces as rubble work in Portland cement mortar.

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* Trans. Am. Soc. C. E., Vol. 48, p. 470.

"In walls of 5 ft. thickness or less, the stones shall average from 6 to 8 cu. ft. each, and the length of the headers shall be two-thirds of the thickness of the wall. In walls over 5 ft. thick the stones shall average 12 cu. ft. each, and headers shall be at least 4 ft. long. Generally, no stones of less than 4 cu. ft. shall be used, except for filling the interstices between the larger stones. At least one-fifth of each face of all the walls shall be composed of headers placed so that those on one face come between those on the other; and, where the thickness of the walls is such that the headers from opposite faces do not overlap, stones not less than 4 ft. long shall be set transversely in the heart of the wall to complete the bond.

"These walls must be built accurately to lines and levels given, and all exposed faces of the stone must be true and straight, and without prominent projections of the quarry faces. The mortar in all joints on exposed faces shall be kept back one inch.

"Selected stone shall be used at all angles, and shall be neatly pitched to true lines and laid on hammer-dressed beds. At all angles 1½-in. draft lines will be required.

"Grout shall be used in place of mortar, wherever directed by the Chief Engineer.

"The face and back of the wall shall be carried up together, over the whole wall, in approximately the same total height, except where otherwise specified.

"In rock-faced work, the faces of the stones shall have uniform projections not exceeding 3 ins., and in rough-pointed work not more than 1 in. beyond the neat lines, and in both cases they shall be pitched to a straight line at all the joints.

"The joints between all stones on the back of all retaining walls and abutments shall be carefully and thoroughly washed with Portland cement mortar so as to make the walls water-tight.

"In cases where rock adjoins the back of the wall, the wall is to be built up tight against the rock, and the joint thoroughly grouted with Portland cement grouting.

" All walls and abutments are to be thoroughly drained, wherever directed by the Chief Engineer, by means of 4-in. cast-iron pipe, built in the wall, or by openings left, as may be ordered. Wherever necessary, tile or French drains of broken stone, 18 × 18 ins., as shown on the drawings, shall be placed at the back of the wall. These drains and pipe will be paid for, per lineal foot, at the price given in the proposal for such work.

" All pointing shall be done with Portland cement mortar of an approved brand. The surfaces of all stone shall be thoroughly cleaned, the joints scraped out to the depth of at least an inch, and the whole thoroughly wetted before pointing is commenced."

Cement.—" In all cases where Portland cement mortar is specified, the mortar shall be composed of 1 part of Portland cement to 3 parts of sand.

" Mortar taken from the mixing box, and molded into briquettes, 1 sq. in. in cross-section, shall develop the following ultimate tensile strength:

AGE.	STRENGTH.
7 days (1 day in air, 6 days in water) 1 part of Portland cement to 3 parts of sand	100 lbs.
28 days (1 day in air, 27 days in water) 1 part of Portland cement to 3 parts of sand	150 lbs.

" Grout made of Portland cement shall be composed of 1 part of Portland cement to 2 parts of sand, except where the foundations are wet, when the quantity of sand shall be diminished, making the proportions 1 part of Portland cement to 1½ parts of sand, which shall be used in the foundation masonry up to the neat lines if required.

" Portland cement shall have a specific gravity of not less than 3, and shall leave, by weight, a residue of not more than 1 per cent on a No. 50 sieve, 10 per cent on a No. 100 sieve, and 30 per cent on a No. 200 sieve.

" Pats of neat cement, ½ in. thick, with thin edges, immersed in water, after 'hard set,' shall show no signs of 'checking' or disintegration.

"It shall require at least 30 minutes to develop initial set.

"Briquettes of cement, 1 sq. in. in cross-section, shall develop the following ultimate tensile strengths:

AGE.	STRENGTH.
24 hours (in water, after hard set)	175 lbs.
7 days (1 day in air, 6 days in water)	450 lbs.
28 days (1 day in air, 27 days in water)	550 lbs.
7 days (1 day in air, 6 days in water), 1 part of cement to 3 parts of standard quartz sand	160 lbs.
28 days (1 day in air, 27 days in water), 1 part of cement to 3 parts of standard quartz sand	220 lbs.

"All cements shall meet such additional requirements as to 'hot water,' 'set,' and 'chemical' tests as the Chief Engineer shall determine. The requirements for 'set' may be modified where the conditions are such as to make it advisable."

Retaining Wall at Black Lick, Ohio.*—The retaining wall shown in Fig. 41 was designed and constructed by Frank A. Bone, of Lebanon, Ohio, for the wing walls of a reinforced concrete arch. A patent has been granted Mr. Bone on this design.

The steel bents shown imbedded near the back of the wall are built up of plates and angles; are placed 4 feet apart and are connected to a 2 × 2-in. angle at the bottom. Small angle brackets are riveted to the upright bents to prevent slipping of the bond. The 4 × ¼-in. plate extending toward the toe is an anchor. The plate is twisted to get a better hold on the concrete. The steel framework is designed to take all the tensile stresses in the wall.

The concrete is composed of 1¼ barrels of cement to ½ cu. yd. of sand and 1 cu. yd. of crushed limestone, with one-third the whole volume of large-sized sound freestone imbedded in the concrete. The walls have a coat of plaster on each side.

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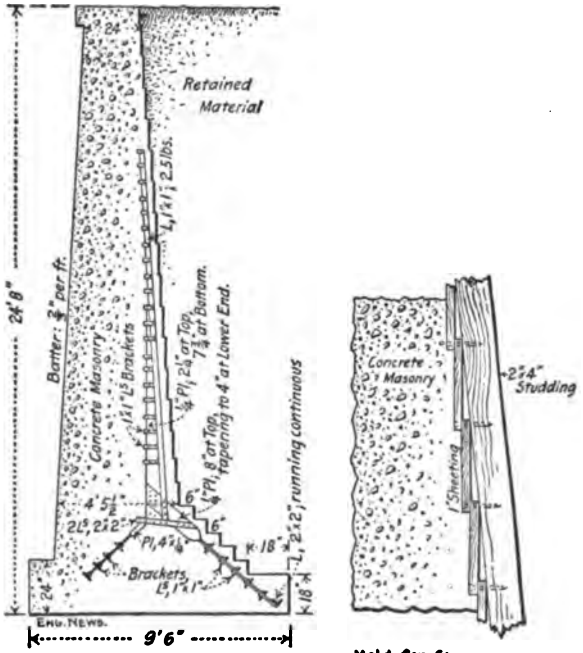
* Engineering News, March, 1902.

The forms for the back of the wall were constructed as shown in Fig. 41.

The same type of wall was in use in Europe as early as 1897. (Engineering News, August 7, 1902.) A retaining wall of this type was also described by David Moliter in the Journal of the Association of Engineering Societies, Vol. XXIV (1890), No. 1. The patent is therefore probably of little value.



Section of Wall.

Mold for Steps in Back of Wall.

Fig. 41.—Reinforced Concrete Retaining Wall at Black Lick, Ohio.

Concrete Retaining Walls at New York Central Terminal, New York.*—The concrete is composed of 1 part Portland cement, 3 parts sand, and 6 parts broken stone. The concrete was mixed by a Ransome mixer. It was mixed very wet and required no ramming. One-man stones were placed in the molds, and the mortar was carefully spaded around the sides of the molds. The wall was made in full height sections 52 ft. long, with open, vertical, transverse, expansion joints ⅛-in. thick between them.

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* Engineering Record, January 6, 1906.

The molds were built with 2-in. pine planks, ship-lapped ½ inch and laid ⅛ inch apart to allow for swelling. They were lined on the face with thin steel plates, and secured by vertical spruce joists and removable horizontal yellow pine waling pieces, bolted on as shown in Fig. 42. The waling pieces were fastened at the corners by angle straps of 2 × ½-inch iron. The longitudinal waling pieces were tied together through the molds with 2 bars ½-in. diameter, placed midway between the ends.

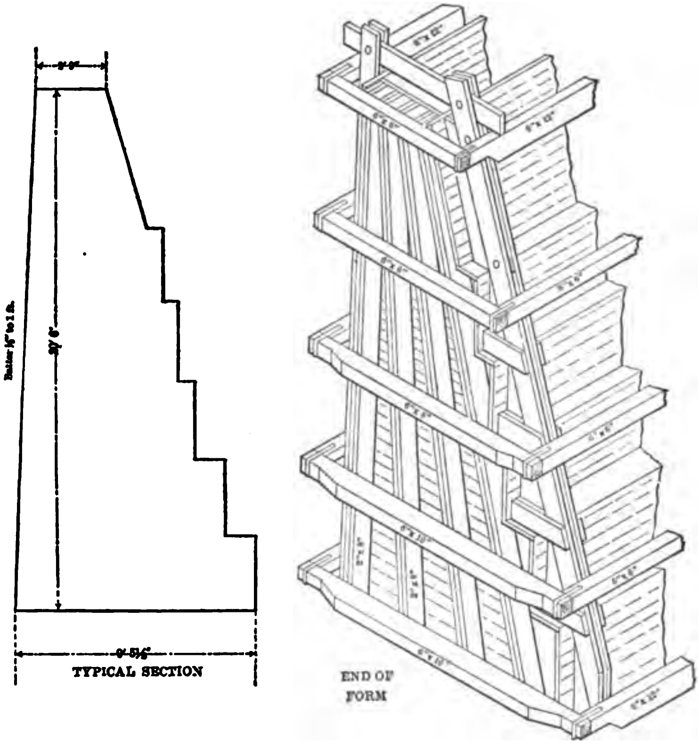
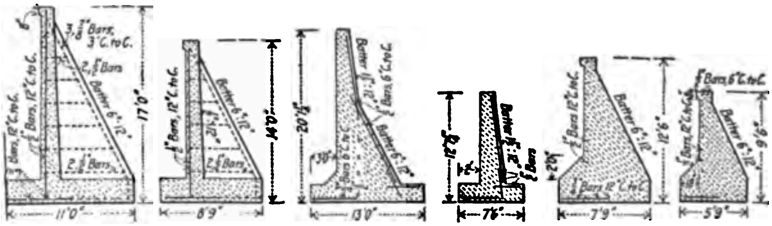


FIG. 42.—CONCRETE RETAINING WALL AND FORM, NEW YORK CENTRAL TUNNEL, NEW YORK.

In placing the concrete three molds were used, one being in use, one standing, and one being erected.

Reinforced Concrete Retaining Walls, C. B. & Q. Ry., Galesburg, Ill.*—The retaining walls are of solid sections up to 12 ft. 6 ins., and beyond that they are of tee and counterfort section. The details of the wall are shown in Fig. 43.



Section A-B Section C-D. Sections of Retaining Walls of East Approach. Section E-F. Section G-H.

FIG. 43.—REINFORCED CONCRETE RETAINING WALLS FOR SUBWAY, GALESBURG, ILL.

Gravel concrete was used, and had the proportions 1 part Portland cement, to 6 parts gravel. The concrete was mixed very wet and was poured like water, thoroughly filling the forms and surrounding the reinforcing bars. The concrete was found to be very dense. The forms were of matched lumber, not lined, and the only attempt at facing the exposed surfaces was to work the face of the concrete with a hoe, so as to press back the stones and show a cement facing.

Reinforced Concrete Retaining Walls for Philadelphia Rapid Transit Co. Tunnel, Philadelphia, Pa.†—The retaining walls were built in 30-ft. sections, with broad footing courses and specially designed reinforcement for each section. The sections forming the portal have ⅝-in. longitudinal horizontal rods on 2 ft. centers, near their central vertical plane, and near the middle of their footing courses to bind them to the walls of the subway, and are the only part of the retaining walls containing reinforcement in those portions. The bulk of the reinforcement in them is in the same position as in the remainder of the

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* Engineering News, February 8, 1906.

† Engineering Record, February 25, 1906.

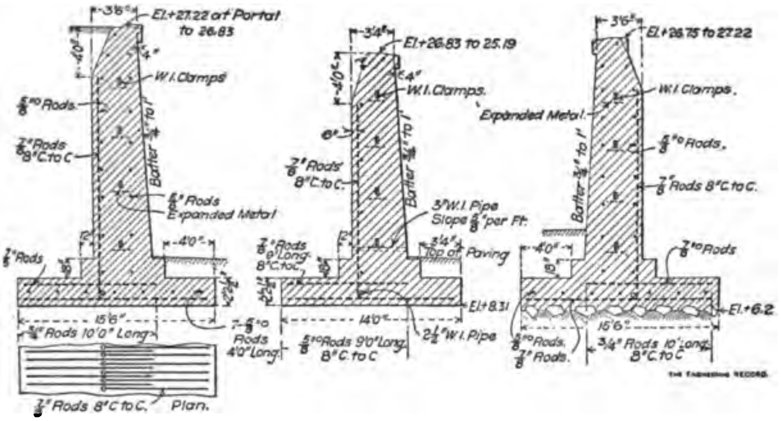

FIG. 44.—REINFORCED CONCRETE RETAINING WALLS FOR PHILADELPHIA RAPID TRANSIT COMPANY SUBWAY.

walls, and is made of ⅞-in. horizontal and vertical bars 6 inches inside their back faces. The horizontal bars are on 2-ft. centers. The vertical bars are 8 inches apart and extend from 4 feet below the top of the wall to within 6 inches of the toe of the footing, where they bend at right angles toward the face and continue horizontal to within 6 inches of the toe of the footing. A piece of 2½ in. gas pipe 7 inches long and filled with concrete is placed in each bend transverse to the rod, and the rod is wired to it to relieve the crushing tendency in the concrete at the point of the bend by an upward pull. Horizontal ¾-in. rods 10 feet long are placed in the same plane with the horizontal portion of the vertical rods, with one end 6 inches from the heel of the footing course, and alternate with those rods on 8-in. centers. Six inches below the top of the lower footing course, which is 2½ feet deep, and of a width of about nine-tenths the height of the wall in a section, are placed transverse horizontal, ⅞-in. rods 10 feet long parallel to the ¾-in. rods below them. Clamps are placed at the joints between the sections which prevent unequal lateral motion of adjoining sections, but permit difference of settlement. The clamps are made of three 5″ × 12″ × ¾″ steel plates. The middle plate is set in one section vertically and the two outside plates are set in the adjacent section. The clamps are spaced 3 feet apart vertically.

The concrete was composed of 1 part Portland cement, 2½ parts gravel or coarse sand, and 5 parts crushed granite or trap rock.

Reinforced Concrete Retaining Wall at Bridge 123, Great Northern Ry.*—The reinforced concrete retaining wall shown in Fig. 45 was designed to hold a sliding embankment at Bridge 123, Kalispel Division, Great Northern Ry. The filling was saturated and rested on a soapstone ledge. The filling was drained by an elaborate system of drains back of the wall. The water was led around the end of the wall, and through holes in the wall. Tamped shell rock filling was deposited back of the wall as shown in Fig. 46. The soapstone was excavated and the wall was placed in a trench as shown in Fig. 46. The concrete was composed of 1 part Portland cement, 2½ parts sand, and 4 parts crushed rock in the wall, and 5 parts crushed rock in the foun-

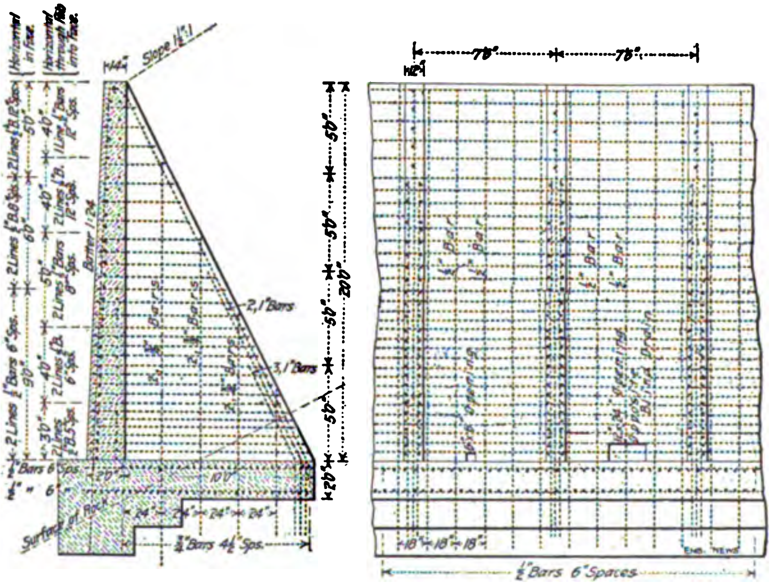

FIG. 45.—REINFORCED CONCRETE RETAINING WALL AT BRIDGE 123.

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* Engineering News, May 6, 1906.

dation. The concrete was mixed very wet so that tamping was not necessary. The facing was put on by means of a steel blade 4″ × 6″ × ¼″ forged on the end of a steel bar 4½ feet long, the rock being simply forced away from the forms with this tool.

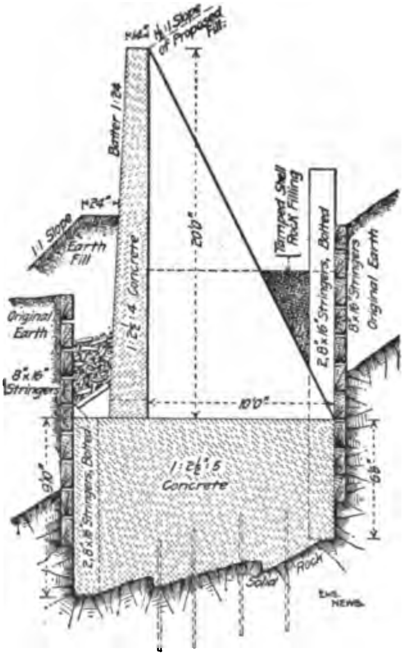

Fig. 46.—Section of Wall Showing Foundation.

The reinforcement consisted of Johnson corrugated bars, arranged as shown in Fig. 45.

To take the pressure of the filling during erection a heavy framing was used in the trench.

A view of the complete wall is shown in Fig. 47.

Hennebique System Retaining Wall.*—The retaining wall shown in Fig. 48 was built at the Paris Exposition of 1900 to support a temporary sunken street near the gardens of Trocadero. The Hennebique System was employed. The retaining wall is divided into

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* Engineering News, February 15, 1900.

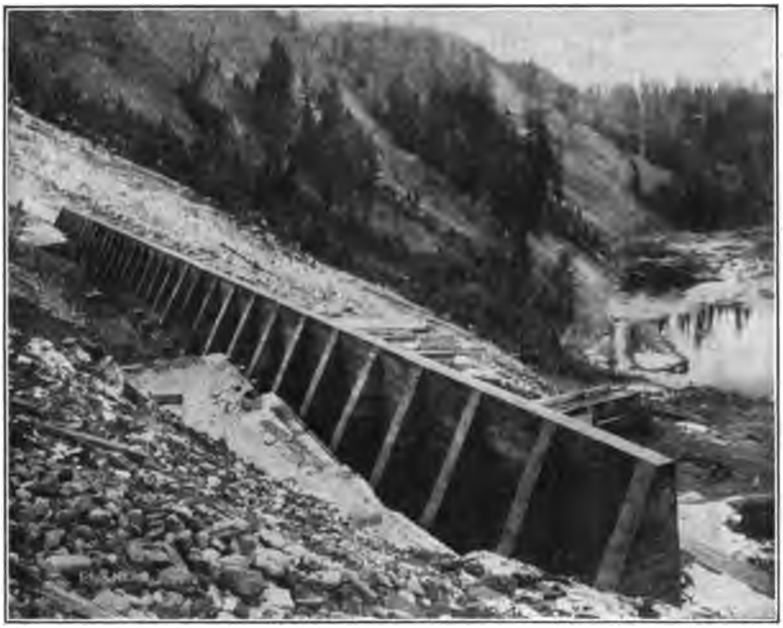

FIG. 47.—REINFORCED CONCRETE RETAINING WALL AT BRIDGE 123.

panels 19.7′ long, and each of these panels is made of reinforced concrete, from the back of which project three buttresses of similar construction. The facing and buttresses are connected by two beams of reinforced concrete. Another buttress projects from the face of the wall and is located a certain distance below the street level. This construction is clearly shown in Fig. 48.

By this arrangement of horizontal beams the earth filling rests on the retaining wall and resists overturning. In the calculation the angle of repose of earth was taken as 35°, and the weight of the earth was taken at 112 lbs. per cu. ft. The allowable pressure on soil at the toe was taken at 2048 lbs. per sq. ft.

The imbedded metal work of the vertical face consists of two series of vertical bars combined with one series of horizontal bars, the spaces, between which, decrease toward the top of the wall. These vertical bars are bent over at right angles at the top to give support for a coping of the same construction as the facing. The reinforcement of the buttresses consist of inclined bars, tied together by straps, and supported by horizontal bars. The horizontal beams are made up of bars in both directions, spaced 5 bars to the square meter; the beams are further strengthened at the edges by flanges. The concrete was of the proportions of 1¼ barrels of cement to 1 cu. yd. of sand and gravel.

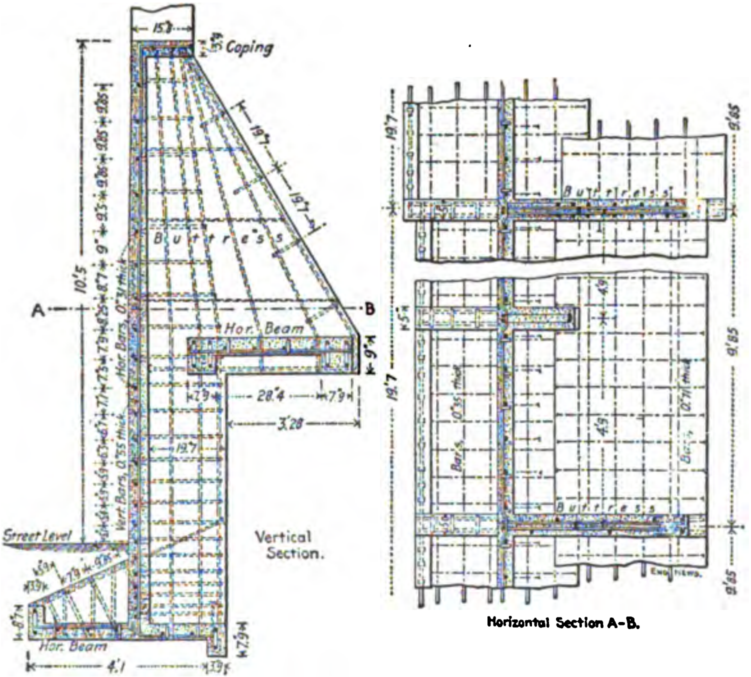

FIG. 48.

Retaining Walls, Railway Terminal Station, Atlanta, Ga.*—The retaining wall to support the embankment approaching the train shed is of novel construction. This type of wall was made necessary owing to the impossibility of securing an adjacent piece of property.

The plan of the wall is given in Fig. 49 and various sections are given in Fig. 50. It will be seen that the usual solid counterforts are replaced by a framework of reinforced concrete beams, ties, and struts. The drawings explain the details of construction.

It will be seen in Fig. 49 that reinforced concrete piles support the

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* Engineering News, April 12, 1906.

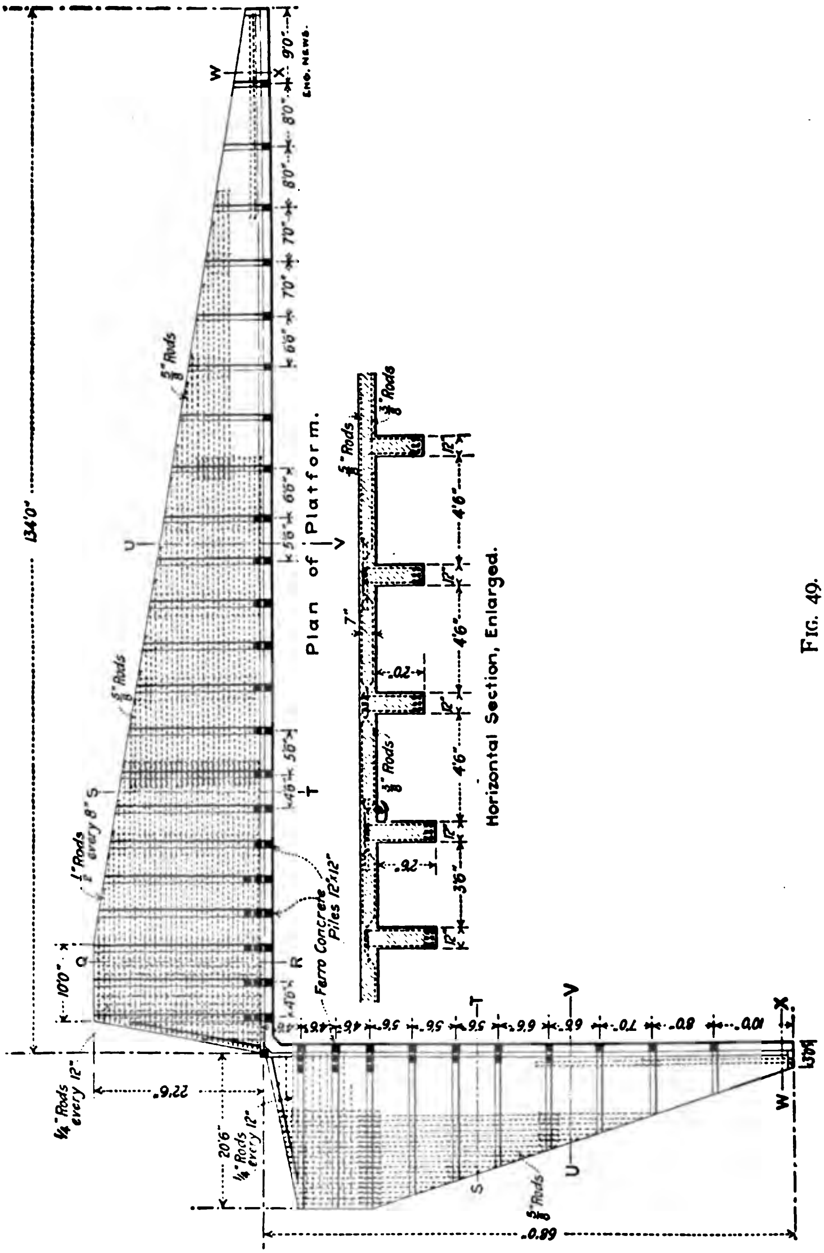
134'0"
Plan of Platform.
Horizontal Section, Enlarged.
Ferro Concrete Piles 12"x12"
5/8" Rods
1" Rods every 8"
1/4" Rods every 12"
Eng. News
9'0"
8'0"
7'0"
6'6"
5'6"
4'6"
10'0"
22'6"
20'6"
68'0"
3'6"
12"
7"
20"
29"
W
X
U
V
S
T
Q
R

Fig. 49.

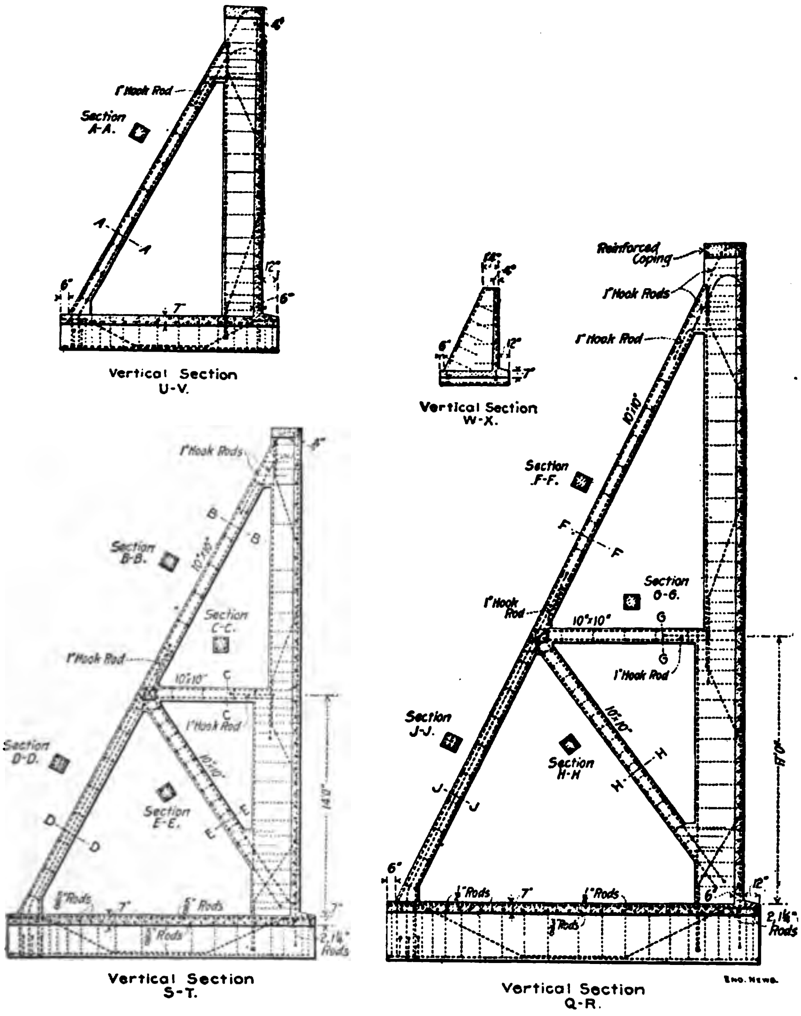

FIG. 50.

toe of the retaining wall. These piles are 12 × 12 ins. square and 10 ft. long. The piles are reinforced with four 1⅛-in. round rods tied up with binders. The piles were concreted horizontally on the ground, and were allowed to set 15 days. They were then driven with an ordinary pile driver weighing 3,500 lbs., dropping 9 feet. The tops were protected with a cast-iron cap with a sawdust cushion between the metal and the concrete.

CHAPTER VII.

COST OF RETAINING WALLS.

Cost of Masonry Retaining Walls.—The cost of masonry walls varies between wide limits, depending upon the cost of stone, cost of quarrying, cost of dressing, cost of laying, cost of mortar, cost of superintendence, cost of tools, cost of maintenance and depreciation of plant. Space will not permit a discussion of all the above items.

Cost of Stone.—The price of stone is usually quoted f. o. b. at the quarry, and varies with the stone and location.

Cost of Quarrying.—After the quarry has been opened in limestone, two-man stone for rubble wall can usually be quarried for from ¼ to ½ the cost of the daily wages of a quarry laborer per cu. yd. Stones ranging from ½ to 1 cu. yd., that have to be blasted, will cost per cu. yd., from ½ to 2 times the cost of the daily wages of one man. Dimension stones that have to be wedged out will cost twice as much as the large stones that can be blasted. This estimate is high for sand stone and low for granite.

Cost of Dressing.—Rubble is roughly scabbled when it is laid and there is no special charge for dressing. Dimension stones, if dressed to lay with quarry finish and fairly close joints, will cost from $1.00 to $3.00 per cu. yd. Bush-hammering costs about 25 cents per sq. ft.

Cost of Laying.—One mason and a helper can lay from 4 to 5 cu. yds. of small rubble in a day of 8 hours. If a derrick is necessary and some dressing required, one mason and a helper will lay only from 2 to 3 cu. yds. of heavy rubble or 1½ to 2 cu. yds. of dimension stone in a day of 8 hours.

Cost of Mortar.—The amount of mortar required varies with the specifications and the stone used. Rubble masonry is from 20 to 35 per cent mortar. Dimension stone masonry is from 10 to 15 per cent mortar. Knowing the cost of cement and sand, the cost of the mortar can be estimated.

Miscellaneous Costs.—The cost of superintendence, tools, maintenance and depreciation of plant, etc., can only be estimated on the particular work. These costs may vary from 5 to 20 per cent of the cost above.

Actual Cost of Stone Masonry Retaining Walls.—The cost of quarrying and laying limestone on the Chicago Sanitary Canal is given by James W. Beardsley as follows:* The limestone occurred in uniform strata, so that the beds required no dressing. The walls were 4 feet wide on top, with a thickness at the bottom of one-half the height, which averaged 24 feet. The stone was laid in courses 12 to 18 inches thick. The stone was scabbled and carefully placed in position on the natural beds, so as to break joints. The stone was laid in

TABLE X.

TABLE SHOWING THE COST OF MASONRY RETAINING WALLS.

Items.	Section 5. Cost per Cubic Yard.	Section 6. Cost per Cubic Yard.	Section 7. Cost per Cubic Yard.	Section 8. Cost per Cubic Yard.	Amount.	Totals. Cost per Cu. Yd.
Wall	$0.900	$1.220	$ 1.240	$1.204	$ 96,701.53	$1.033
Quarry	.770	.750	.340	.862	68,993.47	.738
Cement	.224	.265	.250	.195	22,309.80	.239
Sand	.114	.145	.138	108	11,778.50	.126
Totals	$2.008	$2.380	$1.976	$2.369	$199,783.30	$2.136

	Amount.	Amount.	Amount.	Amount.	Amount.	
Cubic Yards	54,077.8	30,361.72	6,179.9	2,874.9	93,494.3	
Percentage of mortar	12.41	14.8	13.89	10.83	13.25	
Length, right & left, feet.	10,000	10,000	4,676	3,564		
Average height, feet	22.3	15.9	8.8	5.4		
Average area, square feet.	146	82	35.6	21.5		

TYPICAL PLANT EMPLOYED.

Hoisting engine & boiler.	11	8	4	1		
Guy derrick	4	4	2	2		
Stiff leg derrick	7	7	2	1		
Revolving derrick		1				
Steam drills	2	2	1	2		
Air drills		4				
Air hoist		4				
Horse power hoist				2		
Miscellaneous tools	set	set	set	set		
Estimated value	$10,950	$12,825	$4,400	$2,035	$30,120	$0.323
Total cost per cu. yd.						$2.459

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* Journal Western Society of Engineers, December, 1898.

TABLE XI.

MASONRY RETAINING WALLS.

Itemized Table of Force, Percentages and Cost. Based on Sections 5 to 8 Inclusive.

WALL FORCE.

Classification.	Typical Force.	Rates per Day of 10 Hours Each.	Percentage of Cost.	Cost per cu. yd. Cents.
General Foremen	......	$4.50 to $5.00	00.2	00.2
Foremen	1.00	4.00 to 4.50	10.9	11.3
Masons	4.20	3.25 to 3.50	34.2	35.4
Mason's Helpers	1.46	1.50	5.6	5.8
Mortar Mixers	1.81	1.50	7.1	7.3
Mortar Laborers	0.66	1.50	2.6	2.7
Hod Carriers	1.82	1.50	7.1	7.3
Derrickmen	1.77	1.50	6.9	7.1
Enginemen	1.00	2.00 to 2.50	5.2	5.4
Firemen	0.06	1.75	0.3	0.3
Laborers	1.62	1.50	6.3	6.5
Water Boys	0.45	0.75 to 1.00	0.9	0.9
Teams and Carts	0.86	3.50 and 2.50	7.5	7.8
Blacksmiths	0.07	2.00 to 3.00	0.2	0.2
Blacksmith's Helpers	0.06	1.75	0.0	0.1
Carpenters	0.09	2.25 to 2.50	0.5	0.5
Carpenter's Helpers	0.02	1.75	0.0	0.0
Machinists	0.04	3.00 to 4.50	0.3	0.3
Derricks	1.59	1.25 to 1.75	4.0	4.2
Totals	18.58 (men)		99.9	103.3

QUARRY FORCE.

General Foremen	0.01	$4.50 to $5.00	00.2	00.2
Foremen	1.00	3.00 to 4.00	10.6	7.8
Derrickmen	2.11	1.50	10.1	7.5
Quarrymen	8.42	1.65	42.2	31.2
Enginemen	1.10	2.00 to 2.50	7.0	5.2
Firemen	0.04	1.75	0.2	0.2
Laborers	2.28	1.50	10.9	8.0
Water Boys	0.33	0.75 to 1.00	0.9	0.7
Blacksmiths	0.27	2.00 to 3.00	1.7	1.3
Blacksmith's Helpers	0.18	1.75	0.9	0.7
Carpenters	0.02	2.00 to 2.50	0.1	0.0
Carpenter's Helpers	0.00	1.75	0.0	0.0
Drill Runners	0.36	1.75 to 2.00	3.1	2.3
Drill Helpers	0.07	1.50	0.4	0.2
Watchmen	0.04	1.50	0.1	0.1
Machinists	0.00	3.00 to 4.50	0.0	0.0
Teams and Carts	0.29	3.50 and 2.50	3.8	2.8
Derricks	1.12	1.25 to 1.50	5.4	4.0
Drills	0.36	1.00 to 1.25	2.1	1.5
Totals	18.00 (men)		99.7	73.7

mortar composed of 1 part natural cement and 1 part sand. The mortar occupied only 13¼ per cent of the wall. The stone was handled with guy derricks having a capacity of 6 to 10 tons, and a boom 40 to 60 feet long, operated by a hoisting engine. The cost given does not include general superintendence, installation of plant, plant maintenance, power, delays, etc.

Retaining walls were built on four sections, the tabulated costs being as given in Table X and Table XI.

The total cost of the masonry per cubic yard was as follows:

Quarry force	$0.73
Wall force	1.03
Sand, at $1.35 per cu. yd.	0.13
Cement, at $0.60 per bbl.	0.24
Total	$2.13

If the full cost of the plant is charged to the work the cost would be $2.13 + 0.32 = $2.45 per cubic yard. The contract price was $3.25 per cubic yard. The total cost of the masonry in these walls is very low.

Cost of Reinforced Concrete Retaining Walls.—The cost of reinforced concrete may be divided into cost of cement, cost of sand, cost of broken stone, cost of forms, cost of laying, cost of reinforcement, and cost of superintending, etc.

Ingredients in Concrete.—With sand voids 40 per cent, stone voids 45 per cent, and Portland cement barrel holding 3.8 cu. ft., Gillette gives relative quantities of cement, sand and broken stone as in Table XII. The cement is measured packed in the barrel. The quantities for other percentages of voids differ considerably. If the cement is measured loose the quantities of sand and stone are practically the same, but the amount of cement is reduced from 10 to 15 per cent.

Cost of Materials.—The cost of concrete materials varies with the location and must be obtained before preparing an estimate of cost. Portland cement (1906) costs about $1.25 per barrel f. o. b. works;

TABLE XII.*

INGREDIENTS IN 1 CUBIC YARD OF CONCRETE: SAND VOIDS, 40 PER CENT; STONE VOIDS, 45 PER CENT; PORTLAND CEMENT, BARREL YIELDING 3.65 CU. FT. PASTE. BARREL SPECIFIED TO BE 3.8 CU. FT.

Proportions by Volume.	1:2:4	1:2:5	1:2:6	1:2½:5
Barrels Cement per cubic yard Concrete.....	1.46	1.30	1.18	1.13
Cubic yards Sand " " " "	0.41	0.36	0.33	0.40
Cubic yards Stone " " " "	0.82	0.90	1.00	0.80
Proportions by Volume.	**1:2½:6**	**1:3:4**	**1:3:5**	**1:3:6**
Barrels Cement per cubic yard Concrete.....	1.00	1.25	1.13	1.05
Cubic yards Sand " " " "	0.35	0.53	0.48	0.44
Cubic yards Stone " " " "	0.84	0.71	0.80	0.88
Proportions by Volume.	**1:3:7**	**1:4:7**	**1:4:8**	**1:4:9**
Barrels Cement per cubic yard Concrete.....	0.96	0.82	0.77	0.73
Cubic yards Sand " " " "	0.40	0.46	0.43	0.41
Cubic yards Stone " " " "	0.93	0.80	0.80	0.92

Cement is to be measured packed in the barrel.

sand can usually be obtained delivered on wagons at 0.75 to $1.00 per cu. yd.; broken stone can usually be obtained at $1.00 to $1.50 per cu. yd. f. o. b. crusher.

Cost of Forms.—The cost of the timber for forms should be obtained locally. Lumber can be used several times, commonly from 3 to 5 times where forms are torn down each time, and almost indefinitely where forms can be used without change; so that the first cost of lumber should be distributed over all the concrete laid with the forms. The amount of lumber per cubic yard varies with the type of wall, being very much greater for reinforced than for plain walls. A carpenter and a helper should be able to place one M ft. B. M. in two days of 8 hours each, making the cost from $6 to $8 per M ft. B. M.

Cost of Mixing and Placing.—With men at $1.50 per day and a foreman at $3.00 per day, concrete should be mixed and rammed into place for from $0.75 to $1.25 per cu. yd. If the reinforcement is troublesome or the wall narrow the cost may be higher. This does not include facing which will cost more. If machine mixers are used the cost is less on large work.

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*From Gillette's "Cost Data," p. 255.

Actual Cost of Concrete Retaining Walls.—The cost of building concrete retaining walls on the Chicago Sanitary Canal is given by James W. Beardsley as follows:* The walls were 4 feet wide on top, with a thickness at the base of one-half the height, which averaged 10 feet for Section 14, and 22 feet for Section 15. The concrete was 1 natural cement, 1½ sand, and 4 broken stone, faced and coped with 1 : 3 Portland cement mortar 3 inches thick. The costs given do not include general superintendence, installation of plant, plant maintenance, insurance, delays, etc. The Portland cement on Section 14 was 9.3 per cent of the total cement used, and on Section 15, 5.2 per cent.

Section 14.—On Section 14 stone was obtained from the spoil bank of the canal, hauled 100 feet in wheelbarrows to an Austin jaw crusher, which discharged the stone into the bins from which it was fed into a Sooysmith mixer. Both the crusher and the mixer were mounted on a flat car, the cement, sand and stone were raised by a belt elevator so arranged as to discharge into the mixer in the proper proportions. The cement and sand were hauled to the bins in wagons.

Supports for the concrete forms were made by setting vertical posts 9 inches in front of the face of the wall and 8 feet apart, the foot of the post being held in place by a dowel pin set three or four inches into the rock. Wooden braces held the posts in place longitudinally and tie rods connected them with a line of similar posts at the back of the wall. The forms were 16 feet long and 2 feet high. Two one-foot courses were laid, and after an interval of twenty-four hours the forms were drawn, cleaned and reset.

The Portland cement facing was placed by using plates of iron held three inches from the forms by blocks; as the course was completed this plate was withdrawn and the mortar tamped into place. The cost data for Section 14 are given in Table XIII. There were 23,568 cubic yards of concrete masonry on Section 14. The contract prices per cubic yard on Section 14 were: excavation, 38 cents; concrete masonry, $2.74; back filling, 14 cents. Utica cement is rated at 65 cents per bbl.; Portland cement, $2.25 per bbl., and sand at $1.35 per cubic yard.

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* Journal Western Society of Engineers. December, 1898.

TABLE XIII.

CONCRETE RETAINING WALLS. SECTION 14.

Itemized Table of Force, Percentages and Cost.

Nature of Work.	Classification of Labor.	Typical Force.	Rates per Day of 10 Hours Each.	Percentage of Cost.	Cost per Cubic Yard. Cents.
General	Superintendent	1.0	$4.00–$5.00	2.7	2.6
	Blacksmith	1.1	2.50– 3.00	1.7	1.6
	Timekeeper	0.5	2.50	0.7	0.7
	Watchmen	0.6	2.00	0.7	0.7
	Waterboys	3.9	1.00	2.3	2.2
Wall	Foremen	0.9	$2.50	1.3	1.3
	Laborers	8.6	1.50	7.5	7.3
	Tampers	2.3	1.50–$1.75	2.3	2.2
Mixing	Foremen	1.2	$2.50	1.8	1.7
	Enginemen	1.8	2.00–$3.00	2.6	2.5
	Laborers	6.7	1.50	5.8	5.7
	Pump Runners	1.0	1.75–$2.00	1.1	1.0
	Mixers	1.7	1.25	1.2	1.2
Timbering	Foremen	0.6	$2.50	0.8	0.8
	Carpenters	4.7	2.00–$3.00	5.8	5.7
	Laborers	1.2	1.50	1.0	1.0
	Helpers	5.3	2.50	7.7	7.5
Transportation	Foremen	0.0	$2.50	0.0	0.0
	Laborers	2.6	1.75	2.6	2.6
	Teams	6.3	3.00–$3.50	11.9	11.6
Crushing	Foremen	0.5	$2.50	0.7	0.7
	Enginemen	1.7	2.00–$3.00	2.4	2.3
	Laborers	3.5	1.50	3.3	3.2
	Crusher	1.7	1.20	1.1	1.1
Quarrying	Foremen	1.7	$2.50	2.4	2.3
	Laborers	32.9	1.50	28.7	28.0
Totals		94.0 men		100.1	$ .975

MATERIALS

Cement, Utica	$ .863
Cement, Portland	.305
Sand	.465

PLANT VALUES.

2 Crushers	$3,000		
2 Mixers	3,000		
Track	1,260		
Lumber	500		
Pipe	840		
Sheds	400		
Pumps and Boiler	600	$9,600	.407
Total estimated cost per cu. yd			$3.015

Section 15.—The quarry was 1,000 feet from the No. 7 Gates crusher. The rock was hauled to the crusher on 4½ ton-cars by a light locomotive on a standard gage track. A mixer of the spiral screw

TABLE XIV.

CONCRETE RETAINING WALLS, SECTION 15.

Itemized Table of Force, Percentages and Cost.

Nature of Work.	Classification of Labor.	Typical Force.	Rates per Day of 10 Hours Each.	Percentage of Cost.	Cost per Cubic Yard. Cents.
General	Superintendent	1.0	$5.00	2.5	2.4
	Blacksmith	0.9	2.50	1.1	1.1
	Waterboys	4.5	1.00	2.2	2.2
	Teams	1.7	3.00–$3.50	2.6	2.5
Wall	Foreman	1.1	$2.00	1.0	1.0
	Laborers	14.4	1.50	10.6	10.5
	Tampers	0.1	1.50	0.1	0.1
Mixing	Foreman	2.1	$2.50	2.6	2.6
	Enginemen	2.1	2.00–$2.50	2.2	2.2
	Laborers	23.1	1.50– 1.75	18.1	18.0
	Mixers	2.1	2.25	2.2	2.2
Timbering	Carpenters	0.8	$3.00–$3.50	1.3	1.3
	Helpers	10.2	2.50	12.5	12.4
	Laborers	0.7	1.50	0.5	0.5
Transportation	Foremen	0.7	$2.50	0.9	0.9
	Enginemen	1.4	2.00–$3.00	1.9	1.9
	Firemen	0.4	1.50– 1.75	0.3	0.3
	Brakemen	2.2	1.75– 2.00	1.9	1.8
	Teams	0.4	3.00– 3.50	0.7	0.7
	Laborers	1.5	1.50	1.1	1.0
	Locomotives	1.4	2.25	1.5	1.5
Crushing	Foremen	1.0	$2.50–$3.00	1.4	1.4
	Enginemen	1.0	2.50– 3.00	1.4	1.4
	Firemen	1.0	1.50– 1.75	0.8	0.8
	Laborers	11.1	1.50	8.2	8.1
	Crusher	1.0	2.25	1.1	1.1
Quarrying	Foremen	1.0	$2.50–$3.00	1.2	1.2
	Drillmen	1.8	2.00	1.8	1.7
	" Helpers	1.8	1.50	1.3	1.3
	Laborers	19.0	1.50	14.0	13.9
	Drills	1.8	1.25	1.1	1.1
Totals		113.3 (men)		100.1	$.991

MATERIAL.

Powder (quarrying)	$ 0.83
Cement, Utica	.930
Cement, Portland	.180
Sand	.476

ESTIMATED VALUES.

1 Crusher	$12,000		
Use of Locomotive	2,200		
Cars and Track	5,300		
3 Mixers	3,000		
Lumber	1,200		
Pipe	720		
Small Tools	1,000	$25,420	.567
Total estimated cost per cu. yd			$3.227

type was used, operated by an engine and boiler mounted on a separate car. The output of the crusher was about 210 cu. yds. in 10 hours. The output of the mixer was about 100 cu. yds. in 10 hours. The concrete was deposited as on Section 14. The cost data for Section 15 given in Table XIV are on the same basis as those for Section 14. There were 44,811 cubic yards of concrete masonry on Section 15. The contract price for concrete masonry on Section 15 was $3.40 per cubic yard. A comparison of the costs on the two sections is given in Table XV.

TABLE XV.

COMPARISON OF COSTS OF CONCRETE MASONRY ON SECTIONS 14 AND 15, CHICAGO SANITARY CANAL.

	Section 14. Cost per Cubic Yard.	Section 15. Cost per Cubic Yard.		Section 14. Cost per Cubic Yard.	Section 15. Cost per Cubic Yard.
General	$ .078	$ .082	Cement, natural	$ .863	$ .930
Wall	.108	.116	Cement, Portland	.305	.180
Mixing	.121	.250	Sand	.465	.476
Timbering	.150	.142	Plant	.407	.567
Transportation	.142	.081			
Crushing	.073	.128			
Quarrying	.303	.275			
Total	$ .975	$1.074	Total	$3.015	$3.227

Comparison of Plain and Reinforced Concrete Retaining Walls.—(1) In designing a reinforced concrete retaining wall for the Great Northern Ry., Seattle, Wash., Mr. C. F. Graff made a comparative design of plain and reinforced concrete retaining walls. The plain wall had a thickness of 0.4 the height, a front batter of 1 in 12, and was stepped on the back as shown in Fig. 51. The reinforced concrete wall was designed as shown in Fig. 51. With steel at 4½ cents per lb. in place, and concrete at $6.00 per cu. yd. in place, the reinforced concrete wall was 20.4 per cent cheaper for a wall 12 feet high; 36.4 per cent cheaper for a wall 22½ feet high; 43.3 per cent cheaper for a wall 32½ feet high; and 45 per cent cheaper for a wall 42½ feet high.

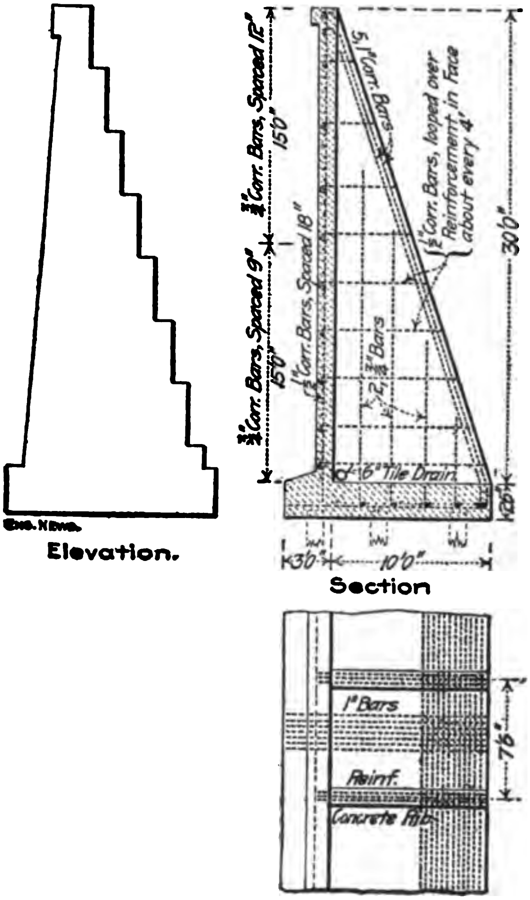


FIG. 51.—PLAIN AND REINFORCED CONCRETE RETAINING WALLS.

(2) In Railroad Gazette, December 30, 1904, Mr. F. F. Sinks gives a comparative design of a plain and a reinforced concrete retaining wall, as shown in Fig. 52. Corrugated bars were used and were

designed for a tensile stress of 12,500 lbs. per sq. in. The following costs include the contractor's profit, but do not include excavation.

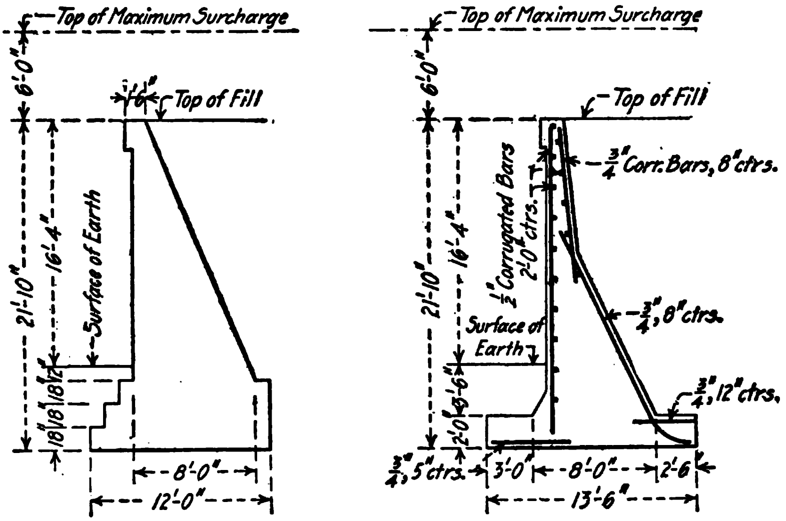


FIG. 52.—DESIGN OF PLAIN AND REINFORCED CONCRETE RETAINING WALLS.

COST OF PLAIN CONCRETE WALL PER LINEAL FOOT IN PLACE.

Concrete—4.8 cu. yds. at $4.00 per cu. yd.	$19.20
Forms—115 ft. B. M. at $31.00 per M.	3.56
Total cost per lineal foot	$22.76

COST OF REINFORCED CONCRETE WALL PER LINEAL FOOT IN PLACE.

Concrete—3.45 cu. yds. at $4.10 per cu. yd.	$14.18
Forms—115 ft. B. M. at $31.00 per M.	3.56
Steel Bars—109 lbs. at $.03¼ per lb.	3.54
Extra Fill—1.34 cu. yds. at $.20 per cu. yd.	.27
Extra Excavation—0.32 cu. yd. at $.20 per cu. yd.	.06
Total cost per lineal foot	$21.61

This shows a saving of $1.15 per lineal foot for the reinforced concrete wall.

PART II.

THE DESIGN OF COAL BINS, ORE BINS, ETC.

Introduction.—Bins are designed to contain materials which are more or less nearly in a granular condition. Dry sand, screened anthracite coal, etc., are almost ideal semi-fluids, while bituminous coal, ashes, etc., are in a variable condition. Coal bins, ore bins, ash bins, etc., will be considered in Part II, while grain bins and grain elevators will be considered in Part III. The bins discussed in Part II are for the greater part shallow bins (bins where the plane of rupture cuts the free surface of the filling), while the bins discussed in Part III are deep bins (bins where the plane of rupture cuts the side of the bin).

CHAPTER VIII.

TYPES OF COAL BINS, ORE BINS, ETC.

Introduction.—Bins for coal, ore, etc., are made of different materials and are of different types. (a) Timber was formerly quite generally used on account of low cost and ease of construction. With more permanent construction fire-proof bins were required and bins are now made (b) of steel with timber cribbed lining, (c) of steel with plate lining, (d) of steel with reinforced concrete lining, and (e) of reinforced concrete.

The material to be confined in the bin is usually delivered into the top of the bin either by dumping directly from cars or by means of conveyors, and is drawn from the bottom through gates, operated by hand levers or by power-driven mechanism. Coal bins, ore bins, etc., are made of several types. The most used are (1) the suspension bunker; (2) the hopper bin; and (3) the circular bin.

1. **Suspension Bunkers.**—Suspension bunkers are made by suspending a steel plate or framework from two side girders; the weight of the filling causing the sides to assume the curve of the equilibrium polygon; the stresses in the plates of a true suspension bunker for a full load being pure tensile stresses.

A Berquist suspension bunker is shown in Fig. 53 and illustrates the method of construction. This bunker is lined with a concrete lining about 1½″ thick, reinforced with wire netting, to protect the metal from the action of the lime.

A suspension bunker constructed of ferroinclave, built by the Brown Hoisting Machinery Co., Cleveland, Ohio, is shown in Fig. 54. The special corrugated sheets are fastened to and are supported by the angle framework. Both sides are then plastered with Portland cement mortar as in making walls and floors (see the Design of Steel Mill Buildings, Chapter XXII).

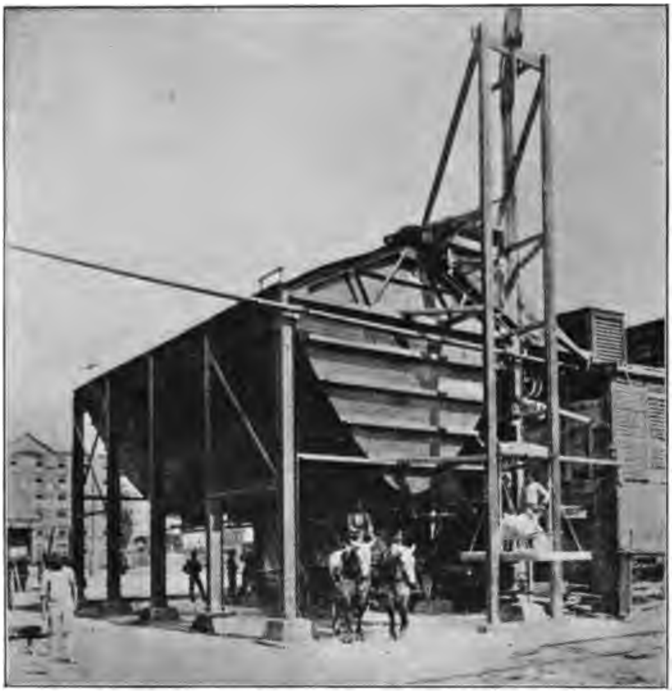


FIG. 53. BERQUIST SUSPENSION BUNKER FOR LIME.

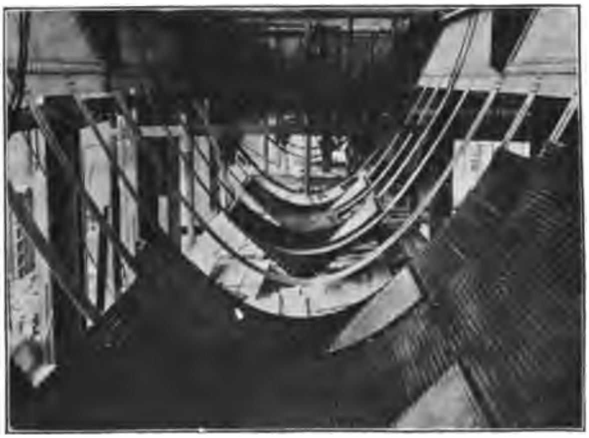


FIG. 54. BROWN HOISTING MACHINERY CO.'S FERROINCLAVE SUSPENSION BUNKER.

For additional examples of suspension bunker bins with complete details see Chapter XII.

2. **Hopper Bins.**—Hopper bins are made of timber, steel, and reinforced concrete. A steel hopper bin built by the Minneapolis Steel & Machinery Co., Minneapolis, Minn., for the Cananea Copper Co., Sonora, Mexico, is shown in Fig. 55. The bins carry the railway tracks, the ore being dumped directly into the bins, then passing through gates to conveyors. Details of these bins are given in Fig. 103, Chapter XII.

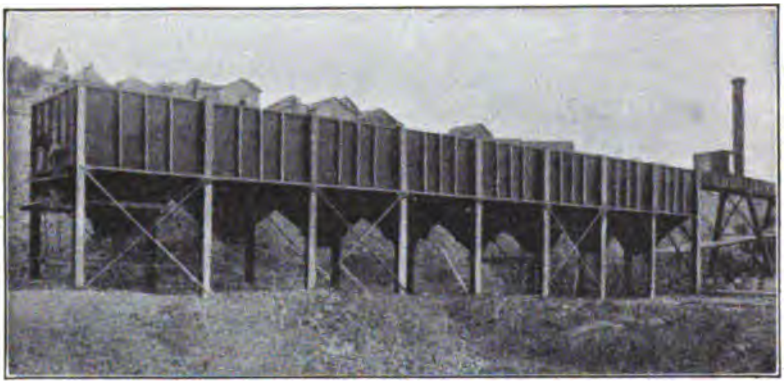
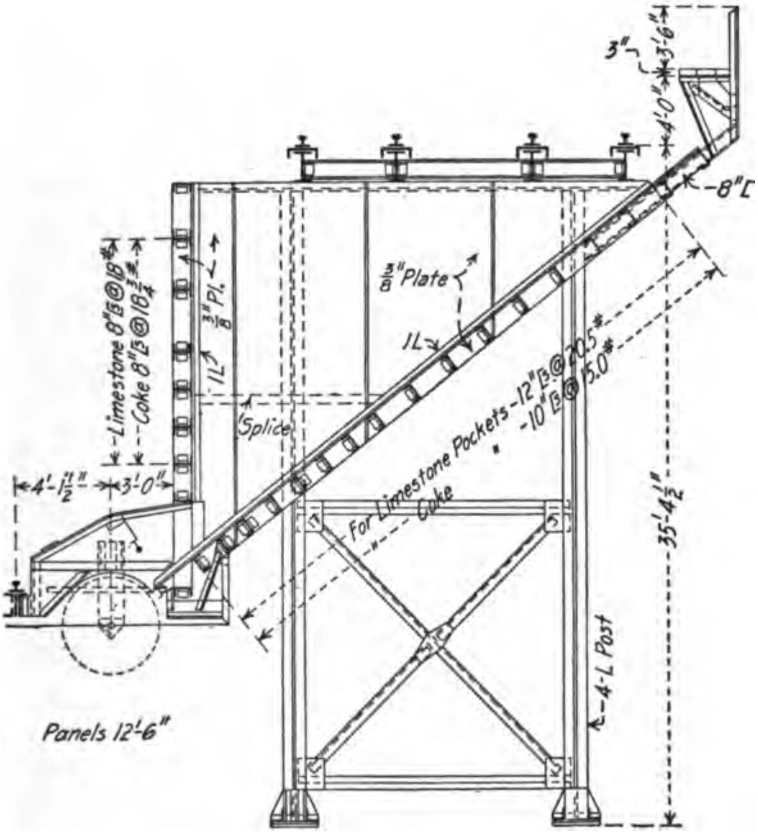


FIG. 55. STEEL HOPPER ORE BIN, CANANEA COPPER CO.

Fig. 56 shows a general cross section of coke and stone bins recently erected for the Lackawanna Steel Company at Buffalo, N. Y. These bins are described by E. W. Pittman in Proc. West. Soc. Penna., June, 1906, as follows:

" These bins are divided into panels of 12′ 6″ center to center of columns, with a double partition at every panel point, leaving the clear length of each pocket about 11′ 6″. The general features of the coke and stone bins are the same, lighter sections being used for the coke bins. Diaphragms are placed between the partitions to prevent the plates from buckling under the action of the normal thrust. These diaphragms also perform the functions of a beam in transmitting the normal thrust to the floor at the bottom and to the transverse channel at the top when one pocket is filled and the adjacent one empty. The bins are lined throughout with ⅜" plates. All rivets in the floor are countersunk.

"Deliveries into these bins are made directly from bottom dumping cars which run on the tracks above, and also from buckets operated from a conveyor bridge.



Typical Section through Pockets.

FIG. 56. COKE AND STONE BINS, LACKAWANNA STEEL CO.

"The gates at the bottoms of the bins are of special construction. They are cylindrical and are revolved by a system of shafting and gears. There is an opening in the side of the drum, and when the drum is revolved this opening comes opposite the opening in the bottom of the bin and the drum is filled. When the drum is further revolved and the opening turned down, its load of material is dumped into a larry for transportation to a furnace skip. At the same time communication between the bin and drum is shut off. The most notable feature of these gates is that they measure the material delivered to the larries. The larries are suspended from the tracks shown in cross section on either side of the gates."

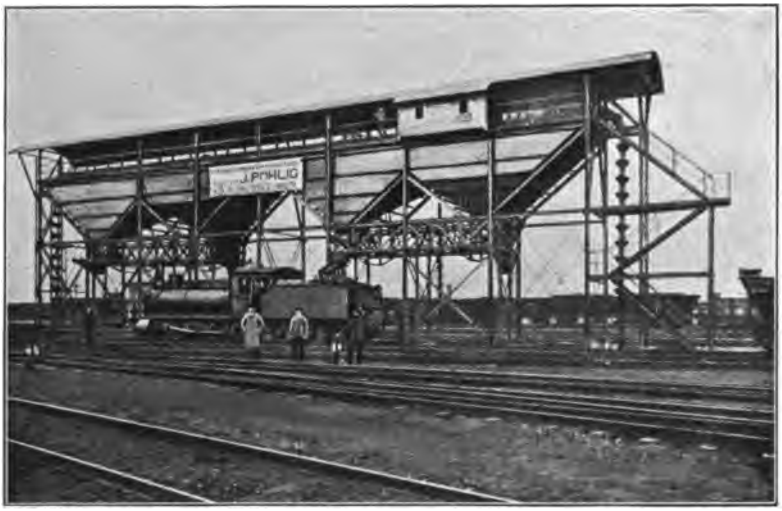


FIG. 57. LOCOMOTIVE COALING STATION AT SAARBRUCKEN, GERMANY.

A locomotive coaling station in which the coal bins are made of steel with hopper bottoms is shown in Fig. 57. For additional examples of hopper bottom bins with complete details see Chapter XII.

3. **Circular Bins.**—Circular bins are made of both steel and reinforced concrete. Steel bins are made with spherical or conical bottoms, while reinforced concrete bins usually have conical bottoms resting on the ground or on columns.

A circular ore bin with a hemispherical bottom, built by the Minneapolis Steel & Machinery Co. at Globe, Arizona, is shown in Fig.

58. Details of this bin are given in Fig. 119 to Fig. 120, Chapter XII.

Another bin made by the same company is shown in Fig. 59. In this bin a masonry wall forms one side of the bin, the other three sides being formed by plates curved as shown and anchored to the masonry.

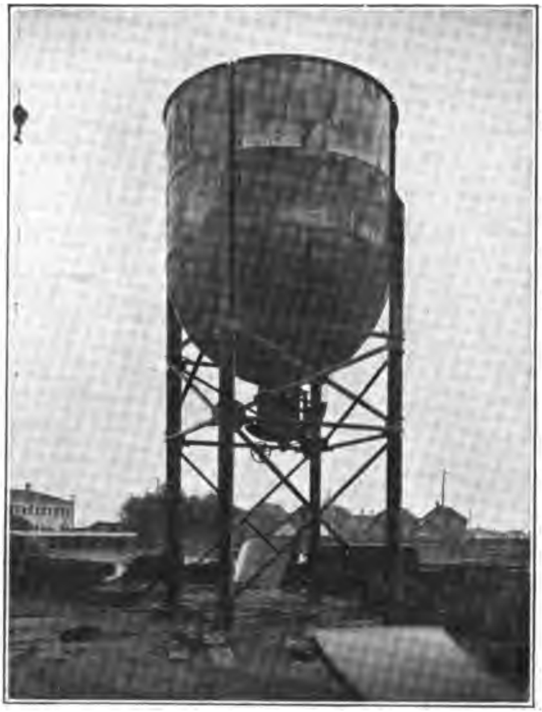


FIG. 58. CIRCULAR ORE BIN, GLOBE, ARIZONA (SEE DETAIL OF GATE IN FIG. 97 b). (Bin set up and photographed at bridge shop.)

Circular steel ore bins have been used in steel rock houses recently constructed for the Ahmeek, the Franklin, and Baltic Mining Companies in the Lake Superior mining district.

For additional examples of circular bins see Chapter XII.

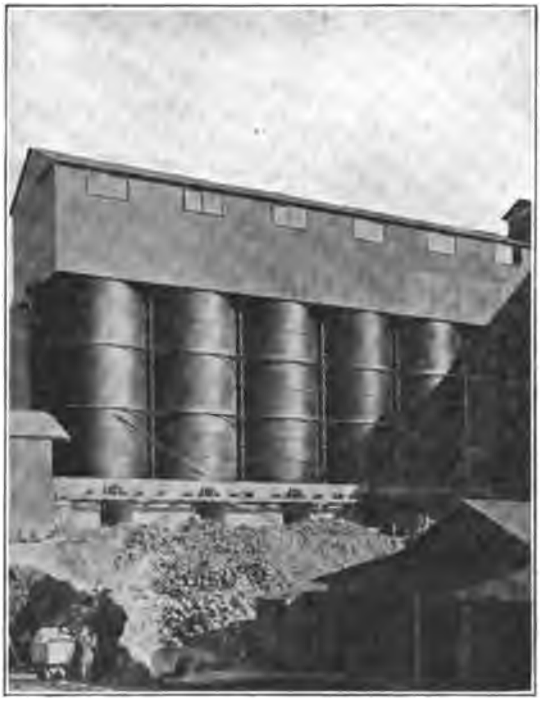


Fig. 59. Steel Ore Bins, Cananea, Mexico.

CHAPTER IX.

STRESSES IN BINS.

Introduction.—In this chapter the stresses will be calculated (1) in bin walls; (2) in hopper bins; (3) in suspension bunkers; (4) in conical bin bottoms; (5) in spherical bin bottoms; (6) in circular girders; and (7) in columns. For the calculation of the stresses in deep bins due to a granular material see Chapter XVI.

1. **STRESSES IN BIN WALLS.**—The problem of the calculation of the pressures on bin walls is the same as the problem of the calculation of pressures on retaining walls. The forces acting on bin walls depend upon the weight, angle of repose, moisture, etc., of the material, which are variable factors, but are less variable than for the filling of retaining walls.

For plane bin walls the formulas developed for retaining walls are directly applicable. The graphic solutions will be found the simplest and most direct for any particular case. Algebraic solutions will be given for six cases which frequently occur in practice. The pressures on plane bin walls will be calculated for bituminous coal, anthracite coal, sand, and ashes, using the data given in an article by R. W. Dull in Engineering News, July 21, 1904. Cases 1 to 6 and Tables XVI to XXII, inclusive, are essentially the same as given in the foregoing article. The same nomenclature will be used as in retaining walls except that P' will be used to indicate the pressure obtained by means of Cain's formulas when $z = \phi'$, N' will indicate the normal component of P', and N will indicate the normal pressure on the wall when $\phi' = 0$. This analysis applies to shallow bins, only.

Case 1. Vertical Wall, Surface Level. Angle $z = \phi'$.

$$P' = \tfrac{1}{2} w . h^2 \frac{\cos^2 \phi}{\cos \phi' \left(1 + \sqrt{\dfrac{\sin (\phi + \phi') \sin \phi}{\cos \phi'}} \right)^2} \tag{63}$$

$$N' = P'.\cos\phi' \tag{64}$$

If $\phi' = \phi$

$$P' = \tfrac{1}{2}w.h^2 \frac{\cos\phi}{(1 + \sin\phi\sqrt{2})^2} \tag{65}$$

$$N' = P'.\cos\phi \tag{66}$$

If $\phi' = 0$, which corresponds to a smooth wall,

$$N = \tfrac{1}{2}w.h^2.\tan^2\left(45^\circ - \frac{\phi}{2}\right) \tag{67}$$

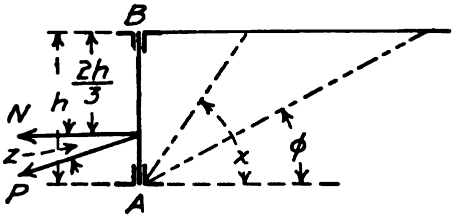


FIG. 60.

TABLE XVI.

CONSTANTS FOR STEEL PLATE BINS, CASE 1.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lbs. Per Cu. Ft.	P' Lbs.	N' Lbs.	N Lbs.
Bituminous coal.	35	18	50	$6.13h^2$	$5.83h^2$	$6.75h^2$
Anthracite coal.	27	16	52	$8.73h^2$	$8.39h^2$	$9.77h^2$
Sand.	34	18	90	$11.50h^2$	$10.93h^2$	$12.72h^2$
Ashes.	40	31	40	$4.02h^2$	$3.44h^2$	$4.34h^2$

Case 2. Vertical Wall, Surface Surcharged at angle δ. Angle $z = \phi'$.

$$P' = \tfrac{1}{2}w.h^2 \frac{\cos^2\phi}{\cos\phi'\left(1 + \sqrt{\frac{\sin(\phi+\phi')\sin(\phi-\delta)}{\cos\phi'.\cos\delta}}\right)^2} \tag{68}$$

$$N' = P'.\cos\phi' \tag{69}$$

If $\delta = \phi$

$$P' = \tfrac{1}{2}w.h^2 \frac{\cos^2\phi}{\cos\phi'} \tag{70}$$

$$N' = P' . \cos \phi' = \tfrac{1}{2} w.h^2 . \cos^2 \phi \tag{71}$$

If

$$\phi' = 0$$

$$N = \tfrac{1}{2} w.h^2 . \cos^2 \phi \tag{72}$$

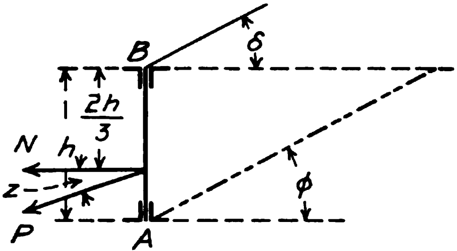


FIG. 61.

TABLE XVII.

CONSTANTS FOR STEEL PLATE BINS, CASE 2. $\delta = \phi$.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lbs. Per Cu. Ft.	P' Lbs.	N' Lbs.	N Lbs.
Bituminous coal.	35	18	50	$17.65h^2$	$16.75h^2$	$16.75h^2$
Anthracite coal.	27	16	52	$21.45h^2$	$20.50h^2$	$20.50h^2$
Sand.	34	18	90	$32.50h^2$	$30.90h^2$	$30.90h^2$
Ashes.	40	31	40	$13.70h^2$	$11.73h^2$	$11.73h^2$

Case 3. Vertical Wall, Surcharge Negative $= \delta$. *Angle* $z = \phi'$.

$$P' = \tfrac{1}{2} w.h^2 \frac{\cos^2 \phi}{\cos \phi' \left(1 + \sqrt{\dfrac{\sin(\phi + \phi') \sin(\phi + \delta)}{\cos \phi' . \cos \delta}}\right)^2} \tag{73}$$

$$N' = P' . \cos \phi' \tag{74}$$

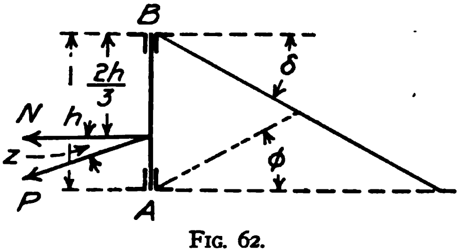


FIG. 62.

If $\phi' = 0$

$$N = \tfrac{1}{2} w.h^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\dfrac{\sin \phi \sin (\phi + \delta)}{\cos \delta}}\right)^2} \tag{75}$$

TABLE XVIII.

CONSTANTS FOR STEEL PLATE BINS, CASE 3. $\delta = -\phi$.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lbs. Per Cu. Ft.	P' Lbs.	N' Lbs.	N Lbs.
Bituminous coal.	35	18	50	$4.49h^2$	$4.27h^2$	$5.13h^2$
Anthracite coal.	27	16	52	$6.64h^2$	$6.38h^2$	$7.64h^2$
Sand.	34	18	90	$8.44h^2$	$8.00h^2$	$9.61h^2$
Ashes.	40	31	40	$2.85h^2$	$2.45h^2$	$3.23h^2$

Case 4. Wall Sloping Outward. $\theta < 90° + \phi'$. *Surface Level.*

$$P' = \tfrac{1}{2} w.h^2 \frac{\sin^2 (\theta - \phi)}{\sin (\phi' + \theta) \sin^2 \theta \left(1 + \sqrt{\dfrac{\sin (\phi + \phi') \sin \phi}{\sin (\phi' + \theta) \sin \theta}}\right)^2}, \tag{76}$$

$$N' = P' . \cos \phi' \tag{77}$$

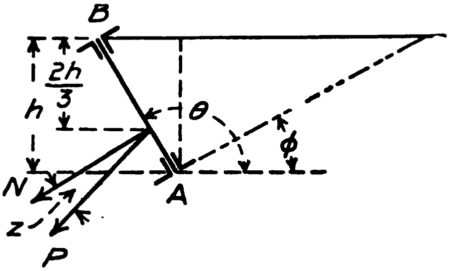


FIG. 63.

Case 5. Wall Sloping Outward. $\theta < 90° + \phi'$. *Surface Surcharged.*

$$P' = \tfrac{1}{2} w.h^2 \frac{\sin^2 (\theta - \phi)}{\sin (\phi' + \theta) \sin^2 \theta \left(1 + \sqrt{\dfrac{\sin (\phi + \phi') \sin (\phi - \delta)}{\sin (\phi' + \theta) \sin (\theta - \delta)}}\right)^2}, \tag{78}$$

$$N' = P' . \cos \phi' \tag{79}$$

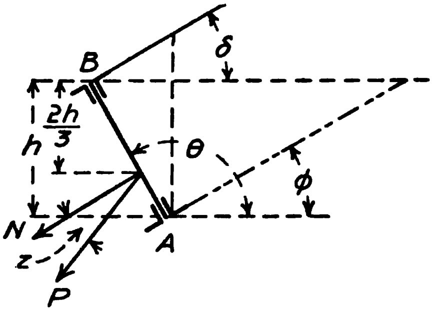


FIG. 64.

Case 6. Wall Sloping Outward. $\theta > 90° + \phi'$. *Surface Level.*

$$P = \tfrac{1}{2} w.h^2 . \tan^2 (45° - \frac{\phi}{2})$$

$$W = weight \ \triangle \ ABC = \frac{w . \tan\theta . h^2}{2}$$

$$E = \sqrt{W^2 + P^2}$$

$$= \tfrac{1}{2} w.h^2 \sqrt{\tan^2\theta + \tan^4 (45° - \frac{\phi}{2})} \qquad (80)$$

$$\tan(\theta + z - 90°) = \frac{\tan\theta}{\tan^2 (45° - \frac{\phi}{2})} \qquad (81)$$

$$Q = E . \cos z \qquad (82)$$

$$T = E . \sin z \qquad (83)$$

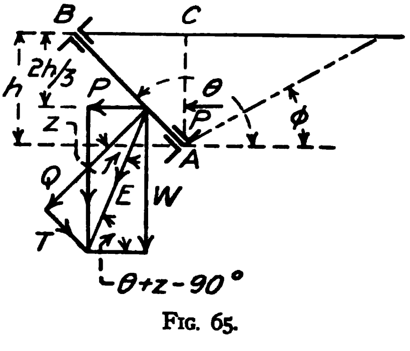


FIG. 65.

For a wall sloping outwards, and sloping surface the use of formulas is cumbersome and the calculations can be more easily made by graphic methods as explained on succeeding pages.

Tables of Pressure on Vertical Bin Walls.*—The normal pressure on vertical bin walls as calculated by the preceding formulas for bituminous coal, anthracite coal, sand, and ashes are given in Table XIX, Table XX, Table XXI, and Table XXII, respectively. In the tables column 1 gives the normal pressure for a smooth vertical wall and horizontal surcharge, while column 4 gives the normal pressure on a rough wall with an angle of friction $=\phi'$. Column 2 gives the normal pressure for a smooth vertical wall and a surcharge $=\phi$, while column 5 gives the normal pressure on a rough wall with an angle of friction $=\phi'$. Column 3 gives the normal pressure for a smooth vertical wall and a negative surcharge $=-\phi$, while column 6 gives the normal pressure on a rough wall with an angle of friction $=\phi'$. It will be seen that the pressures in columns 2 and 5 are identical. For a vertical wall with $\delta=\phi$, the normal pressures as given by Rankine's and Cain's formulas are identical. To prove this substitute $\delta=\phi$ in (8), multiply by $\cos\phi$ and compare with (41).

The total pressures are given for a wall one foot long in all cases.

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* Link Belt Machinery Co.'s Standards; R. W. Dull, Engineering News, July 21, 1904.

TABLE XIX.

Total Pressure in Pounds for Depth "h" for Bituminous Coal.
Wall One Foot Long.

$w = 50$ lbs., $\phi = 35°$.

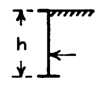
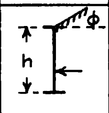
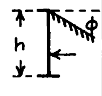
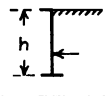
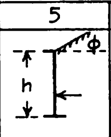
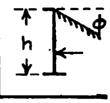
Depth h in Feet	Smooth Wall, $\phi' = 0$			Rough Wall, Angle of Friction $= \phi'$		
	1	2	3	4	5	6
						
	$\phi' = 0$	$\delta = \phi$	$\delta = -\phi$	$\phi' = 18°$	$\delta = \phi$	$\delta = -\phi$
1	6.75	16.75	5.13	5.83	16.75	4.27
2	27	67	20.5	23.32	67	17.1
3	60.75	150.75	46.2	52.47	150.75	38.4
4	108	268	82	93.4	268	68.3
5	168.75	418.75	128	145.7	418.75	107
6	243	603	184.5	209.4	603	156
7	333	821	257	286	821	209
8	432	1,072	328	373	1,072	273
9	547	1,357	415	472	1,357	346
10	675	1,675	513	583	1,675	427
11	817	2,027	615	705	2,027	516
12	972	2,412	738	840	2,412	615
13	1,141	2,831	866	985	2,831	722
14	1,323	3,283	1,005	1,143	3,283	838
15	1,519	3,769	1,152	1,312	3,769	960
16	1,728	4,288	1,311	1,492	4,288	1,093
17	1,951	4,841	1,480	1,685	4,841	1,232
18	2,187	5,427	1,660	1,889	5,427	1,382
19	2,437	6,047	1,852	2,105	6,047	1,541
20	2,700	6,700	2,052	2,332	6,700	1,708
21	2,977	7,387	2,262	2,571	7,387	1,883
22	3,267	8,102	2,483	2,821	8,102	2,067
23	3,571	8,861	2,560	3,084	8,861	2,259
24	3,888	9,648	2,810	3,358	9,648	2,460
25	4,219	10,469	3,206	3,644	10,469	2,669
26	4,563	11,323	3,468	3,941	11,323	2,887
27	4,923	12,211	3,740	4,250	12,211	3,113
28	5,292	13,142	4,022	4,570	13,142	3,348
29	5,677	14,087	4,314	4,903	14,087	3,591
30	6,075	15,075	4,617	5,247	15,075	3,843

TABLE XX.

TOTAL PRESSURE IN POUNDS FOR DEPTH "h" FOR ANTHRACITE COAL. Wall One Foot Long.

$w = 52$ lbs., $\phi = 27°$.

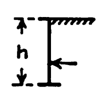
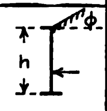
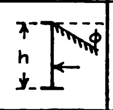
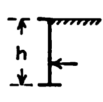
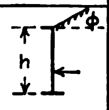
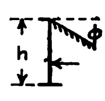
Depth h in Feet	Smooth Wall, $\phi'=0$			Rough Wall, Angle of Friction $=\phi'$		
	1	2	3	4	5	6
						
	$\phi'=0$	$\delta=\phi$	$\delta=-\phi$	$\phi'=16°$	$\delta=\phi$	$\delta=-\phi$
1	9.75	20.5	7.64	8.39	20.5	6.38
2	39.0	82.0	30.6	33.5	82.0	25.5
3	87.8	184.5	68.8	75.5	184.5	57.5
4	156	328	122.2	134.2	328	102.0
5	244	513	191	210	513	159.5
6	351	738	267	302	738	230
7	478	1,005	374	411	1,005	313
8	624	1,312	489	536	1,312	402
9	790	1,661	619	680	1,661	517
10	975	2,050	764	839	2,050	638
11	1,180	2,481	925	1,014	2,481	773
12	1,405	2,952	1,100	1,209	2,952	920
13	1,648	3,465	1,290	1,418	3,465	1,080
14	1,910	4,018	1,497	1,643	4,018	1,250
15	2,193	4,613	1,720	1,887	4,613	1,436
16	2,500	5,248	1,953	2,145	5,248	1,636
17	2,808	5,945	2,207	2,421	5,945	1,845
18	3,160	6,642	2,471	2,718	6,642	2,064
19	3,521	7,400	2,758	3,030	7,400	2,310
20	3,902	8,200	3,053	3,350	8,200	2,554
21	4,303	9,041	3,372	3,700	9,041	2,820
22	4,718	9,922	3,701	4,061	9,922	3,086
23	5,156	10,845	4,040	4,438	10,845	3,372
24	5,611	11,808	4,398	4,833	11,808	3,680
25	6,097	12,813	4,770	5,244	12,813	3,985
26	6,600	13,858	5,160	5,672	13,858	4,521
27	7,112	14,945	5,560	6,116	14,945	4,650
28	7,638	16,072	5,979	6,578	16,072	5,000
29	8,202	17,241	6,421	7,056	17,241	5,370
30	8,775	18,450	6,880	7,551	18,450	5,742

TABLE XXI.

TOTAL PRESSURE IN POUNDS FOR DEPTH "h" FOR SAND.
Wall One Foot Long.

$w = 90$ lbs., $\phi = 34°$.

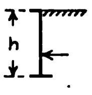
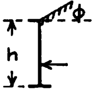
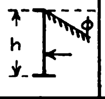
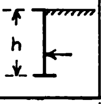
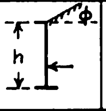
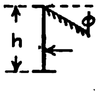
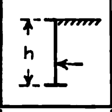
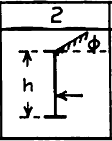
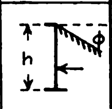
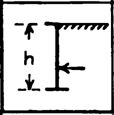
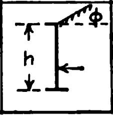
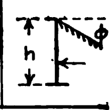
Depth h in Feet	Smooth Wall, $\phi'=0$			Rough Wall, Angle of Friction $=\phi'$		
	1	2	3	4	5	6
						
	$\phi'=0$	$\delta=\phi$	$\delta=-\phi$	$\phi'=18°$	$\delta=\phi$	$\delta=-\phi$
1	12.72	30.9	9.61	10.93	30.9	8
2	50.8	123.6	38.4	43.7	123.6	32
3	114.5	278	86.40	98.5	278	72
4	203.7	494	113.8	175	494	128
5	318	772	240	273	772	200
6	458	1,113	346	394	1,113	288
7	624	1,515	471	535	1,515	392
8	815	1,980	615	700	1,980	512
9	1,030	2,500	778	885	2,500	648
10	1,272	3,090	961	1,093	3,090	800
11	1,540	3,740	1,162	1,345	3,740	968
12	1,833	4,450	1,383	1,575	4,450	1,152
13	2,150	5,230	1,624	1,848	5,230	1,352
14	2,495	6,060	1,880	2,160	6,060	1,568
15	2,862	6,960	2,160	2,460	6,960	1,800
16	3,256	7,910	2,460	2,798	7,910	2,048
17	3,676	8,930	2,777	3,159	8,930	2,312
18	4,121	10,012	3,114	3,541	10,012	2,592
19	4,592	11,155	3,469	3,946	11,155	2,888
20	5,088	12,360	3,844	4,372	12,360	3,200
21	5,610	13,627	4,238	4,820	13,627	3,528
22	6,156	14,956	4,651	5,290	14,956	3,872
23	6,729	16,346	5,084	5,782	16,346	4,232
24	7,327	17,798	5,535	6,296	17,798	4,608
25	7,950	19,313	6,006	6,831	19,313	5,000
26	8,599	20,889	6,496	7,389	20,889	5,408
27	9,273	22,526	7,006	7,968	22,526	5,832
28	9,972	24,225	7,534	8,569	24,225	6,272
29	10,698	25,987	8,082	9,192	25,987	6,728
30	11,448	27,810	8,649	9,837	27,810	7,200

TABLE XXII.

Total Pressure in Pounds for Depth "h" for Ashes.
Wall One Foot Long.
$w = 40$ lbs., $\phi = 40°$.

Depth h in Feet	Smooth Wall, ϕ' = 0			Rough Wall, Angle of Friction = ϕ'		
	1	2	3	4	5	6
						
	$\phi' = 0$	$\delta = \phi$	$\delta = -\phi$	$\phi' = 31°$	$\delta = \phi$	$\delta = -\phi$
1	4.35	11.73	3.23	3.44	11.73	2.45
2	17.4	47	12.9	13.76	47	9.80
3	39.2	105.7	29.01	30.96	105.7	22.05
4	69.6	188	31.7	55.04	188	39.20
5	108.7	294	80.8	86	294	61.2
6	156.4	423	116	124	423	88.2
7	213	576	158	168	576	120
8	278	751	207	220	751	157
9	352	952	261	279	952	199
10	435	1,173	323	344	1,173	245
11	526	1,420	391	416	1,420	296
12	626	1,690	465	495	1,690	353
13	735	1,985	546	581	1,985	414
14	852	2,300	634	674	2,300	480
15	978	2,640	726	774	2,640	550
16	1,113	3,010	828	881	3,010	627
17	1,257	3,400	934	994	3,400	708
18	1,408	3,803	1,045	1,115	3,803	794
19	1,527	4,240	1,165	1,242	4,240	884
20	1,740	4,700	1,290	1,376	4,700	980
21	1,920	5,181	1,423	1,517	5,181	1,080
22	2,100	5,677	1,561	1,665	5,677	1,186
23	2,300	6,215	1,706	1,820	6,215	1,296
24	2,506	6,756	1,860	1,981	6,756	1,411
25	2,720	7,331	2,017	2,150	7,331	1,531
26	2,940	7,929	2,180	2,325	7,929	1,656
27	3,165	8,551	2,352	2,508	8,551	1,786
28	3,406	9,196	2,530	2,697	9,196	1,921
29	3,660	9,865	2,718	2,893	9,865	2,060
30	3,915	10,557	2,910	3,096	10,557	2,205

Pressure Exerted by Coal Against Vertical Retaining Walls, per Foot of Length by Trautwine's Formulas.—The Link Belt Engineering Co. uses Trautwine's formulas for calculating the pressure of coal. The solutions and tables for anthracite and bituminous coal based on these formulas follow:

Anthracite Coal.—Angle of repose of coal = 27°.

Weight of anthracite coal per cubic foot = 52.1 lbs.

For Surface Horizontal.—Fig. 66.

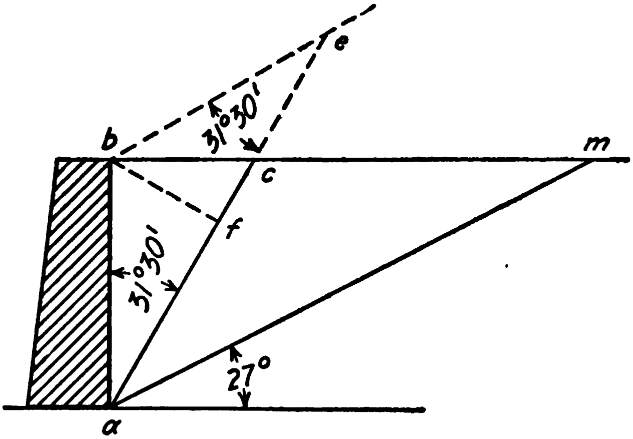


FIG. 66.

$$\text{Horizontal pressure} = \frac{\text{weight of the triangle } acb \times bc}{ab}$$

$$= \frac{52.1 \times ab \times bc \times bc}{2ab} = 26.05\,(bc)^2, \quad (84)$$

or, the presure varies directly as the square of the depth, d.

When the Surface Slope is Equal to the Angle of Repose.—Fig. 66.

$$\text{Horizontal pressure} = \frac{\text{weight of the triangle } aeb \times bc}{ab}$$

$$= \frac{52.1 \times bf \times af \times bc}{ab} = 52.1\,(bf)^2, \quad (85)$$

again, the pressure varies directly as the square of the depth, d.

TABLE XXIII.

PRESSURE OF ANTHRACITE COAL ON VERTICAL BIN WALLS CALCULATED BY TRAUTWINE'S FORMULAS.*

Depth in Ft.	Horizontal Surface.		Sloping Surface.		Depth in Ft.	Horizontal Surface.		Sloping Surface.	
	Total Pressure.	Pressure on Lowest Foot.	Total Pressure.	Pressure on Lowest Foot.		Total Pressure.	Pressure on Lowest Foot.	Total Pressure.	Pressure on Lowest Foot.
1	9.78	9.78	14.22	14.22	26	6,611	498	9,612	725
2	39	29	56.88	42.66	27	7,129	518	10,366	753
3	88	48	127.98	71	28	7,667	537	11,149	782
4	156	68	227	99	29	8,225	557	11,988	810
5	244	88	355	127	30	8,802	577	12,797	839
6	352	107	511	156	31	9,398	596	13,665	867
7	479	127	696	184	32	10,015	616	14,561	895
8	625	146	910	213	33	10,650	635	15,486	924
9	792	166	1,151	241	34	11,306	655	16,439	952
10	978	185	1,422	270	35	11,980	674	17,420	981
11	1,183	205	1,720	298	36	12,675	694	18,429	1,009
12	1,408	224	2,047	327	37	13,389	713	19,467	1,038
13	1,652	244	2,403	355	38	14,123	733	20,533	1,066
14	1,916	264	2,787	383	39	14,875	753	21,629	1,095
15	2,200	283	3,199	412	40	15,648	772	22,752	1,123
16	2,503	303	3,640	440	41	16,440	792	23,904	1,151
17	2,826	322	4,109	469	42	17,252	811	25,084	1,180
18	3,168	342	4,607	497	43	18,083	830	26,293	1,208
19	3,530	361	5,133	526	44	18,934	850	27,530	1,237
20	3,912	381	5,688	554	45	19,804	870	28,793	1,265
21	4,313	400	6,271	583	46	20,695	889	30,090	1,294
22	4,733	420	6,882	611	47	21,605	909	31,412	1,322
22	5,173	440	7,522	639	48	22,533	929	32,763	1,350
23	5,633	459	8,190	668	49	23,482	948	34,143	1,379
45	6,112	479	8,887	696	50	24,450	968	35,550	1,407

Surface horizontal { Total pressure $= 9.78\,d^2$.
Pressure on lowest foot $= 9.78(2d - 1)$.

Surface sloping { Total pressure $= 14.22\,d^2$.
Pressure on lowest foot $= 14.22(2d - 1)$.

Bituminous Coal.—Angle of repose of bituminous coal = 35°. Weight of bituminous coal per cubic foot = 47 lbs.

From Fig. 67 in the same manner as for anthracite coal we have:

Surface horizontal { Total pressure $= 6.37\,d^2$.
Pressure on lowest foot $= 6.37(2d - 1)$.

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* Link Belt Engineering Co.

TABLE XXIV.

PRESSURE OF BITUMINOUS COAL ON VERTICAL BIN WALLS CALCULATED BY TRAUTWINE'S FORMULAS.*

Depth in Ft.	Horizontal Surface.		Sloping Surface.		Depth in Ft.	Horizontal Surface.		Sloping Surface.	
	Total Pressure.	Pressure on Lowest Foot.	Total Pressure.	Pressure on Lowest Foot.		Total Pressure.	Pressure on Lowest Foot.	Total Pressure.	Pressure on Lowest Foot.
1	6.4	6.4	10	10	26	4,305	325	6,760	510
2	25	19	40	30	27	4,641	338	7,290	530
3	57	32	90	50	28	4,993	350	7,840	550
4	102	45	160	70	29	5,358	363	8,410	570
5	159	57	250	90	30	5,732	376	9,000	590
6	229	70	360	110	31	6,122	389	9,610	610
7	312	83	490	130	32	6,523	401	10,240	630
8	407	96	640	150	33	6,935	414	10,890	650
9	516	108	810	170	34	7,362	427	11,560	670
10	637	121	1,000	190	35	7,778	440	12,250	690
11	770	134	1,210	210	36	8,253	452	12,960	710
12	917	146	1,440	230	37	8,754	465	13,690	730
13	1,076	159	1,690	250	38	9,193	478	14,440	750
14	1,248	172	1,960	270	39	9,682	490	15,210	770
15	1,433	185	2,250	290	40	10,192	503	16,000	790
16	1,630	197	2,560	310	41	10,669	516	16,810	810
17	1,840	210	2,890	330	42	11,236	529	17,640	830
18	2,063	223	3,240	350	43	11,797	541	18,490	850
19	2,298	236	3,610	370	44	12,331	554	19,360	870
20	2,548	248	4,000	390	45	12,968	567	20,250	890
21	2,809	261	4,410	410	46	13,478	580	21,160	910
22	3,083	274	4,840	430	47	14,100	592	22,090	930
23	3,369	287	5,290	450	48	14,679	605	23,040	950
24	3,669	299	5,760	470	49	15,275	618	24,010	970
25	3,981	312	6,250	490	50	15,925	631	25,000	990

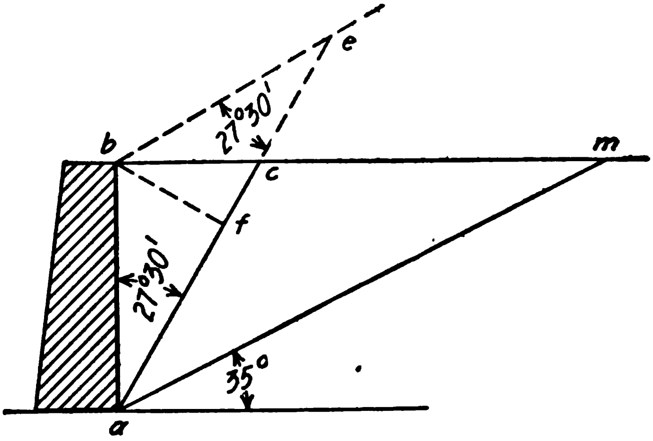

FIG. 67.

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* Link Belt Engineering Co.

$$\text{Surface sloping}\quad \begin{cases} \text{Total pressure} = 10\,d^2. \\ \text{Pressure on lowest foot} = 10(2d - 1). \end{cases}$$

The pressures on vertical walls based on the above formulas are given for anthracite coal in Table XXIII and for bituminous coal in Table XXIV.

2. **STRESSES IN HOPPER BINS.**—The stresses in hopper bins can be most easily calculated by graphic methods or by a combination of algebraic and graphic methods. In the following solutions the bins are shallow so that the angle of rupture cuts the top surface of the material within the bin, and the bin walls are assumed as smooth. The following cases will be considered: (1) top surface level; (2) top surface heaped; (3) top surface surcharged. The bin and data are assumed practically as in the Paterson, N. J., bin, the failure of which in 1897 brought out a spirited discussion in Engineering News.

1. **Hopper Bin, Top Surface Level.**—The bin is 32′ 0″ wide, 18′ 0″ high, and the pockets are 17′ 0″ long. The bin will be assumed to be filled with coal weighing 58 lbs. per cu. ft., and having an angle of

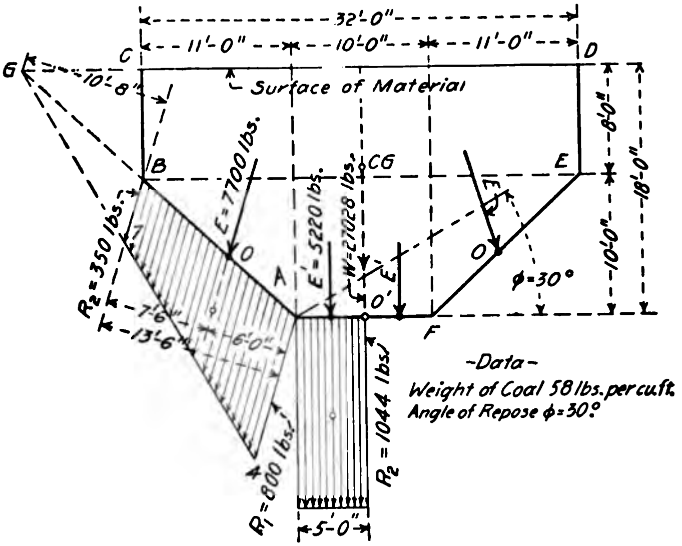


Fig. 68.

repose of $\phi = 30°$. (Anthracite coal is commonly assumed to weigh 52.1 lbs. per cu. ft., and to have an angle of repose of 27°.) The analysis will be made for a section of the bin 1 ft. long.

(*a*) *First Method.*—In Fig. 68 it is required to calculate the pressure on the inclined side *A-B*. The direction of maximum pressure in the filling will be vertical and will be $p = w.y = 58 \times 18 = 1044$ lbs. per sq. ft. This is the unit pressure on the bottom *A-F*, and the

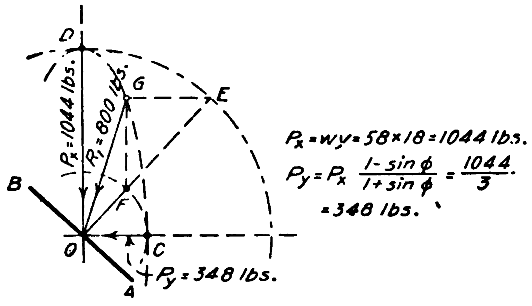

D
G
E
B
O
F
C
A
$P_x = 1044$ lbs.
$R_1 = 800$ lbs.
$P_x = w.y = 58 \times 18 = 1044$ lbs.
$P_y = P_x \frac{1-\sin\phi}{1+\sin\phi} = \frac{1044}{3}$.
$= 348$ lbs.
$P_y = 348$ lbs.

FIG. 69.

resultant pressure on the left half of the bottom will be $E' = 1044 \times 5 = 5220$ lbs. To calculate the pressure on the side *A-B*, construct the ellipse of stress at the point *A*, as in Fig. 69, and determine the unit pressure on the plane *A-B* at *A*. The ellipse of stress at point *A* is constructed in Fig. 69 as follows: Lay off $OD = 1044$ lbs.; and $OC = 1044 \times \frac{1 - \sin\phi}{1 + \sin\phi} = 348$ lbs. Now draw *AB* in Fig. 69, and draw *OF* normal to *A-B*. Construct circles with radii *OD* and *OC*, then *G* is a point on the ellipse of stress, by construction, and $OG = R_1 = 800$ lbs. = the pressure on the plane *A-B* in Fig. 68. Now in Fig. 68 lay off *A*4 to represent the unit stress, R_1, at *A* on the plane *A-B*, and draw line 4*G*. The pressure at *B* on the plane *A-B* will be 350 lbs. per sq. ft., and may be calculated from the pressure area or calculated by an ellipse of stress as in Fig. 69. The total pressure on the side *A-B* will be equal to the area of the shaded load polygon and will act through the center of gravity of the area as shown, and is $E = 7{,}700$ lbs. The pressure on side *C-B* at *B* may be calculated by means of the ellipse of stress and is $=8\times 58\times \frac{1-\sin\phi}{1+\sin\phi}=155$ lbs. per sq. ft.

(*b*) *Second Method.*—In Fig. 70 the total pressure on the plane A-H is $P_1=\frac{1}{2}w.h^2\,\frac{1-\sin\phi}{1+\sin\phi}=3{,}130$ lbs., acting horizontally through a point 12′ 0″ below the top surface. Now to find the pressure P_2 on the plane G-A produce P_1 until it intersects the line $O2=$ weight of triangle $AHG=10{,}440$ lbs. at O, and by construction $O1=P_2=$ 10,900 lbs. P_2 is parallel to E in Fig. 70. The normal pressure on A-G is 9,900 lbs. Now 9,900 lbs. acts through the center of gravity of triangle $AG4$, and is equal to the area of $AG4$. The normal pressure at A is 733 lbs. per sq. ft. and the normal pressure at B is 320 lbs. per sq. ft., and the normal pressure on A-B acts through the center of gravity of the shaded area and is $N=7{,}200$ lbs. Also by construction $E=7{,}700$ lbs. as before. The pressures on bottom A-F and wall B-C are found as in the first solution. Solutions (*a*) and (*b*) give identical results.

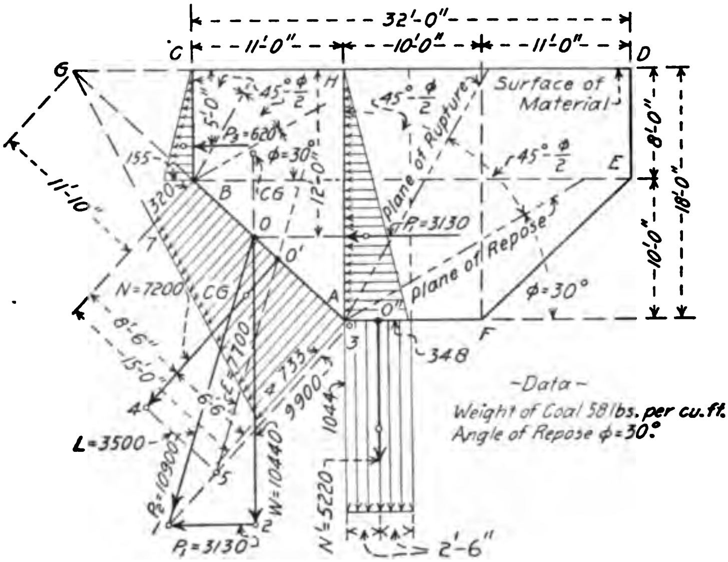
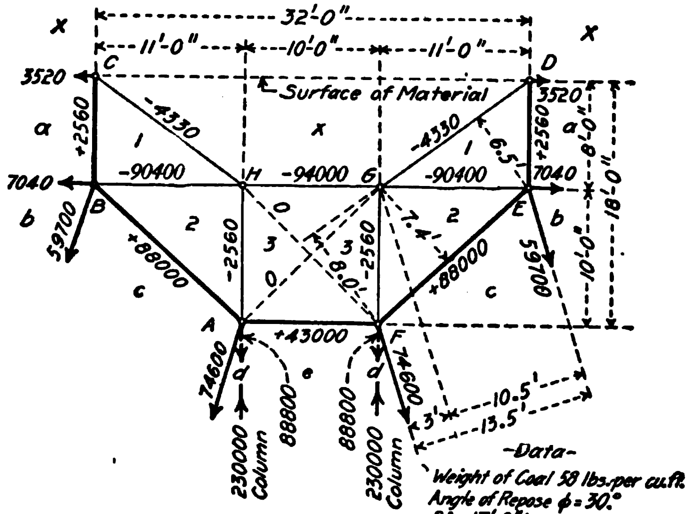


FIG. 70.



-Data-
Weight of Coal 58 lbs. per cu.ft.
Angle of Repose $\phi = 30°$
Bin 17'-0" long.

FIG. 71 a.

Algebraic Moments.

Center of Moments, E.

Stress GD.

-GD × 6.5' - 3520 × 8' = 0

GD = -4330

Stress FG.

-FG × 11' - 3520 × 8' = 0

FG = -2560

Center of Moments, F.

Stress GH.

-GH × 10' - 3520 × 18' - 7040 × 10' - 59700 × 13.5' = 0

GH = -94000

Stress GE.

-GE × 10' + 4330 × 8' - 3520 × 18' - 7040 × 10' - 59700 × 13.5 = 0 GE = -90400

Center of Moments, G.

Stress ED.

-ED × 11' + 3520 × 8' = 0

ED = +2560

Stress FE.

FE × 7.4' - 3520 × 8' - 59700 × 10.5 = 0

FE = +88000

Stress AF.

AF × 10' + 74600 × 3' - 59700 × 10.5' - 3520 × 8' = 0 AF = +43000

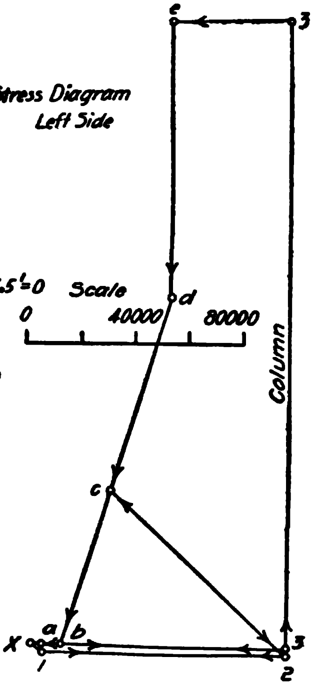


FIG. 71 b.

Calculation of Stresses in Framework.—The loads on the bin walls are carried by transverse framework as in Fig. 71, spaced 17′ 0″ center to center. The loads at the joints act parallel to the pressures as previously calculated, the loads being found in the same manner as for a simple beam loaded with a similar loading. To calculate the stresses by graphic resolution in Fig. 71 b, lay off loads x-a = 3,520 lbs., a-b = 7,040 lbs., b-c = 59,700 lbs., c-d = 74,600 lbs., and d-e = 88,000 lbs. Then beginning at joint C construct the diagram in the usual manner. The stresses may also be calculated by algebraic moments as explained in Fig. 71 b.

Center of Gravity of a Trapezoid.—To calculate the center of gravity of the trapezoid 1–2–3–4 in Fig. 72 proceed as follows: Bisect the parallel sides 1–4 and 2–3 and draw bisecting line a-b. Now produce side 3–2 to the left a distance B = length of side 1–4, and produce side 1–4 to the right a distance A = length of side 2–3; then connect the ends of the lines 2–3 and 1–4 produced, and the intersection of this line with the line a-b will give the center of gravity of the trapezoid.

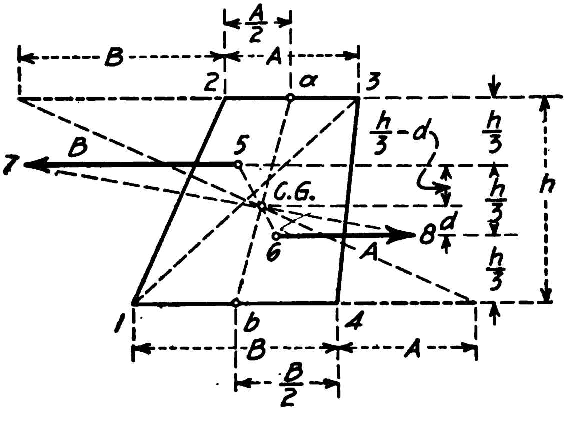


FIG. 72.

Proof.—To obtain the center of gravity of trapezoid 1–2–3–4 divide it into the triangles 1–2–3 and 1–3–4. Then the area of the triangles 1–2–3 and 1–3–4 may be represented by the lines A and B, respectively.

Now assume that B acts to the left through 5, the center of gravity of triangle 1–2–3, and A acts to the right through 6, the center of gravity of triangle 1–3–4. Then line 7–8 will cut line 5–6 at the center of gravity of the trapezoid 1–2–3–4. Let $\frac{h}{3}+d$ be the distance of the center of gravity above the base. It is required to prove that the dotted line joining 3–2 and 1–4 produced cuts the line a-b at a distance $\frac{h}{3}+d$ above the base. Now in Fig. 72 we have in the similar triangles the bases are to each other as the altitudes, and

$$A+\frac{B}{2}:B+\frac{A}{2}::x:h-x,$$

where x is the altitude of the lower triangle. Solving

$$h.A+\frac{h.B}{2}-A.x-\frac{B.x}{2}=B.x+\frac{A.x}{2}$$

and

$$x=\frac{(2A+B)h}{3(A+B)} \qquad \text{(a)}$$

$$=\frac{Ah}{3(A+B)}+\frac{h}{3}\frac{(A+B)}{(A+B)} \qquad \text{(b)}$$

But in triangles 8–6–CG, and 7–5–CG,

$$A:B::d:\frac{h}{3}-d$$

and

$$d=\frac{A.h}{3(A+B)} \qquad \text{(c)}$$

from which we have

$$x=d+\frac{h}{3} \qquad \text{(d)}$$

which proves the construction.

2. **Hopper Bin, Top Surface Heaped.**—The bin in Fig. 73 is heaped at the angle of repose, $\phi=30°$. To calculate the pressure on side A-B, proceed as follows: Locate points G and H, and calculate the horizontal pressure $P_1=7{,}680$ lbs., acting on the plane H-K at

$\frac{1}{3}HK$ above H. Pressure P_1 was calculated by the graphic method described in Chapter II, Fig. 23. Produce P_1 until it intersects at O the line of action of the weight of the triangle GHK acting through the center of gravity of the triangle. From O lay off $O1 = W =$ 19,900 lbs., acting downwards, and from 1 lay off $1\text{-}2 = P_1 = 7{,}680$ lbs., acting to the left. Then $O2 = P_2 = 21{,}300$ lbs. Now $P_2 =$ area triangle $6GH$, and $E =$ area $8\text{-}B\text{-}A\text{-}5 = 11{,}340$ lbs. Force E acts through the center of gravity of area $8\text{-}B\text{-}A\text{-}5$. The horizontal pressure on plane $C\text{-}B = 1{,}400$ lbs. $=$ area $s'e'n'.w$. The vertical pressure on the left-hand side of the bottom $A\text{-}F$ is 7,400 lbs., acting through the center of gravity of the pressure polygon. The vertical unit pressure at A is 1,440 lbs. per sq. ft.

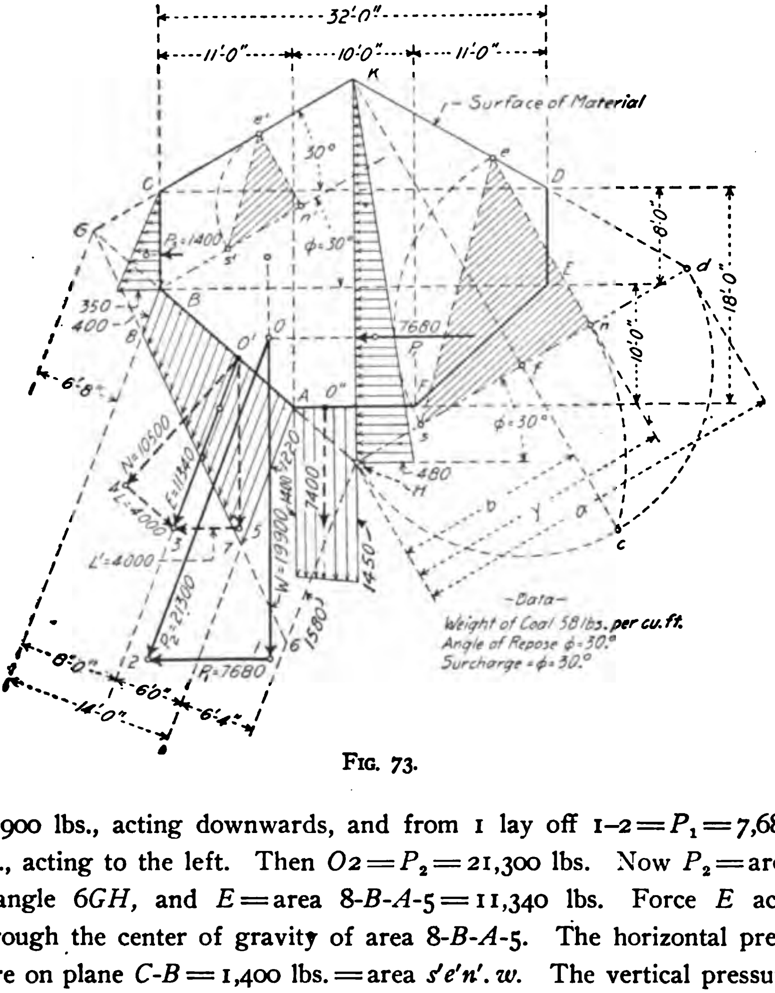


FIG. 73.

Calculation of Stresses in Framework.—The stresses in a transverse truss are given in Fig. 74. The stresses are calculated in the same manner as in Fig. 71 b. It will be seen that the stresses in several of the members are very much larger than when the top surface is level as in Fig. 71 a.

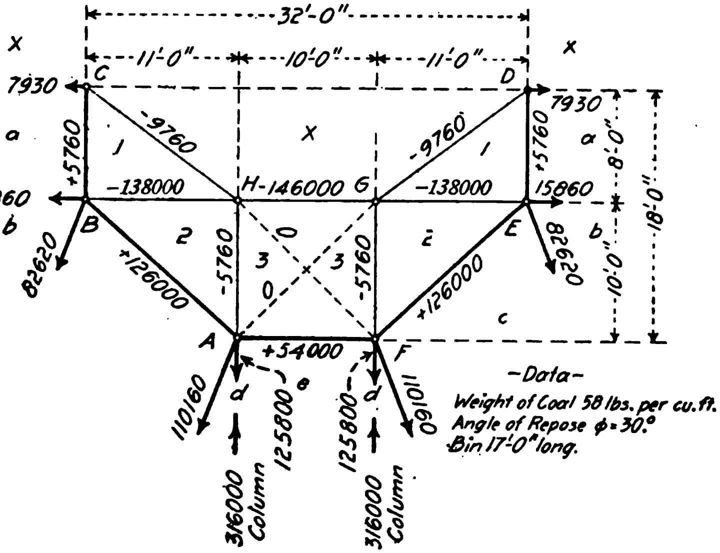


Fig. 74.

3a. **Hopper Bin, Top Surface Surcharged.**—The bin in Fig. 75 is surcharged from the left to the right side, the surcharge being equal to ϕ, the angle of repose. The pressure on the bin wall *C-B* was calculated as in Fig. 73, while the presure on bin wall *E-D* was calculated as on *K-H* in Fig. 73.

To calculate the presure on bin bottom *A-B* the pressure on plane *H-K* was calculated as in Fig. 21, and was found to be $P_1 = 22{,}060$ lbs. To calculate the pressure on *G-H* produce P_1 to *O*, and P_2 is the resultant of P_1 and *W*, the weight of the triangle *HGK*, and is $P_2 = 30{,}000$ lbs. Using the same method as in Case 2, *E* is found to act through the center of gravity of area *7-B-A-6*, and to be equal to 15,500 lbs. To calculate the pressure on bin bottom *F-E* the pressure on plane *H-K* was calculated as in Case 2, and was found to be

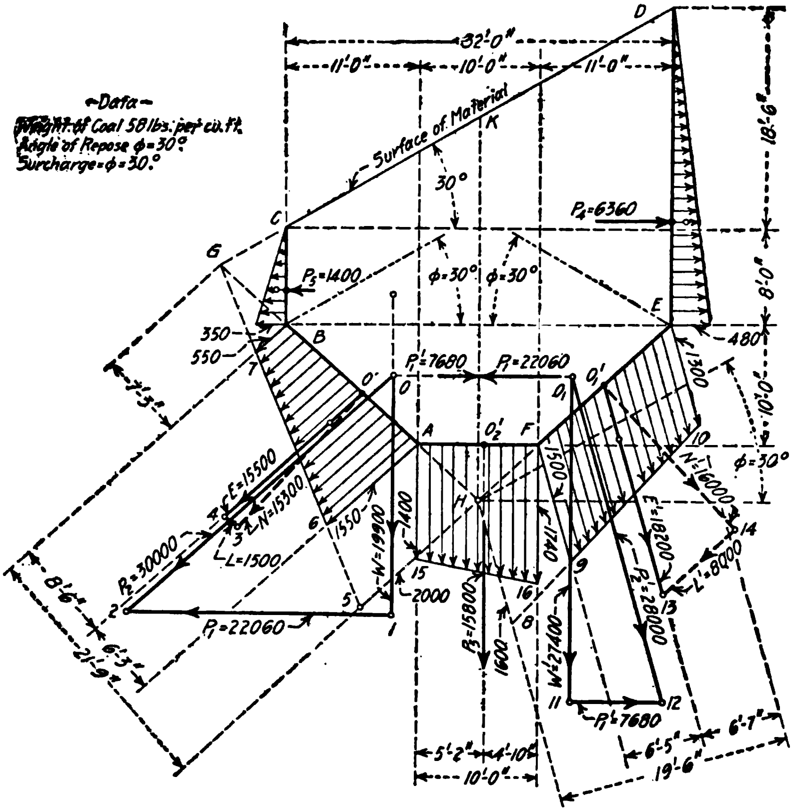
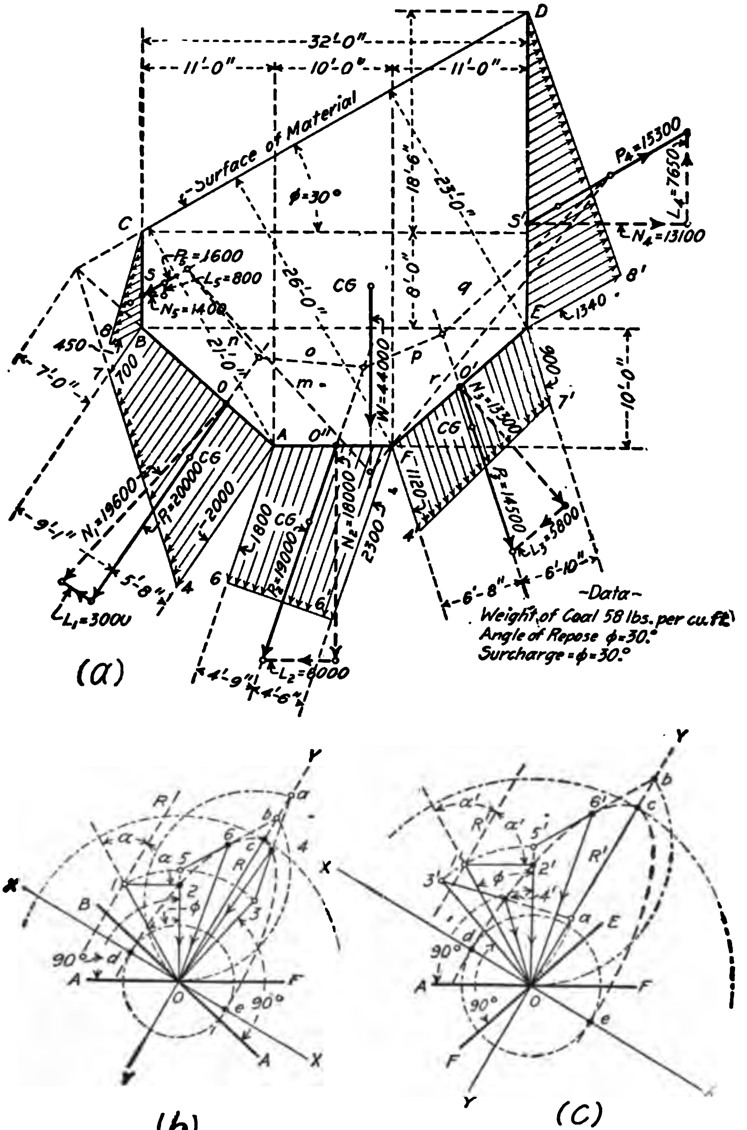


FIG. 75.

$P_1' = 7{,}680$ lbs. To calculate the pressure on *H*-*E* produce P_1' to O_1, and P_2' is the resultant of P_1' and W', the weight of the area *H*-*K*-*D*-*E*, and is $P_2' = 28{,}000$ lbs. Now P_2' acts through the center of gravity of area 8-*H*-*E*-10, and is equal to the area. By calculation we find $E' = 18{,}200$ lbs. and acts through the center of gravity of area 9-*F*-*E*-10. The stresses in the framework may be calculated as in Case 2.

3b. **Hopper Bin, Top Surface Surcharged and of Indefinite Extent.**—The bin in Fig. 76 is surcharged from the left to the right side, the surcharge being equal to ϕ, the angle of repose, the material extending indefinitely to the right. (This condition seldom occurs in practice.)



(a)

(b) (c)

FIG. 76.

The conditions of this problem entirely satisfy Rankine's definition of a granular mass and the stresses may be calculated by the ellipse of stress as given in (*b*) and (*c*). The pressures at point *A* are calculated in (*b*) as follows: In (*b*) lay off $O1$ normal to the plane of repose and equal to $21 \times 58 = 1{,}218$ lbs. Draw $O5$ vertical and describe arc 15. Then point 2 is a point on the ellipse of stress. Through 1 draw $1R$, making angle $\alpha =$ angle α, and through O draw OY parallel to $1R$. Then Oc is the semi-major axis and Od the semi-minor axis of the ellipse of stress. To calculate the stress on side *A-B*, draw $O3 = O1$, normal to *A-B* and with point 3 as a center and a radius equal $O3$ describe an arc cutting OY at points O and *a*. Lay off $34 = 12$, and $O4 = 2{,}000$ lbs. is the pressure per square foot on plane *A-B* at point *A*. By proportion the pressure at *B* is 700 lbs. per sq. ft., and the total pressure is $P_1 = 20{,}000$ lbs. acting through the center of gravity of 7-*B*-*A*-4. To calculate the pressure on bottom *A-F* at *A*, draw $O5 = O1$ normal to *A-F* in (*b*), with radius $O5$ and center 5 describe an arc cutting OY at *b*, lay off $56 = 12$, and $6O = 1{,}800$ lbs. = pressure per square foot at *A* in (*a*).

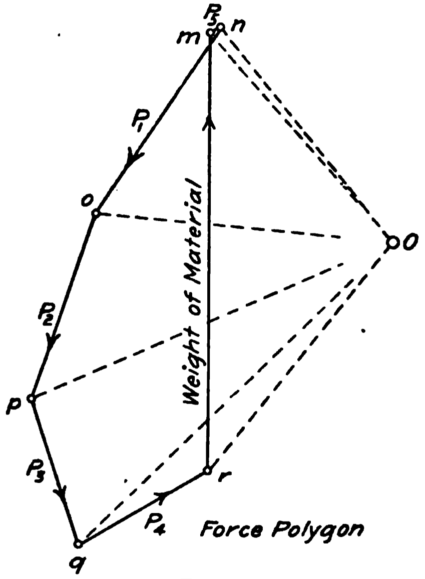


FIG. 77.

The pressures on *F-E* and *A-F* at *F* are calculated in (*c*) in the same manner. And $P_2 = 19{,}000$ lbs., and $P_3 = 14{,}500$ lbs. By means of a similar ellipse of stress the pressure at E is found to be 1,340 lbs. per sq. ft., and at B is 450 lbs. per sq. ft., and $P_4 = 15{,}300$ lbs., and $P_5 = 1{,}600$ lbs. Pressure P_4 is the passive resistance and is very much in excess of the maximum active pressure. However, if the force polygon for the forces P_1, P_2, P_3, P_4, P_5 and the weight of material be constructed as in Fig. 77 the force polygon will be found to close. The stresses are those that would occur when the bin carries the coal and also takes the thrust from a mass of coal extending up to the right beyond the bin proper. This condition seldom occurs and the problem is of scarcely more than academic interest.

3. **STRESSES IN SUSPENSION BUNKERS.**—The suspension bunker shown in (a) Fig. 78 carries a load which varies from zero at the support to a maximum at the center. If the bunker is level full

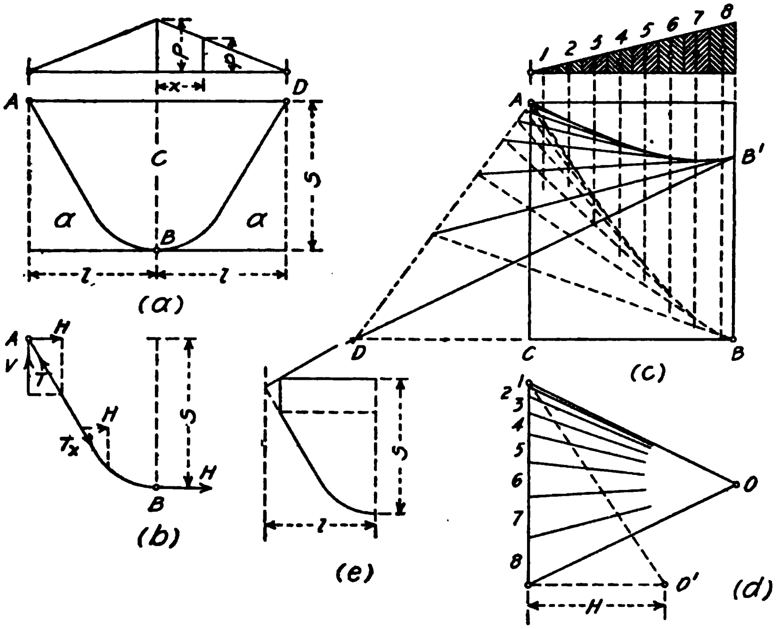


FIG. 78.

the loading from the supports to the center varies nearly as the ordinates to a straight line, while if the bunker is surcharged the straight line assumption for loading is more nearly correct.

We will, therefore, assume that the loading of the bunker in (*a*) is represented by the triangular loading varying from $p =$ zero at each support to a maximum of $p = P$ at the center.

Let $l =$ one-half the span,
$S =$ the sag,
$H =$ the horizontal component of the stress in the plate,
$T =$ maximum tension in plate,
$V =$ reaction of the bunker,
$C =$ capacity of bunker,
$B =$ origin of coördinates.

Now if the right-hand half of the bunker be cut away as in (*b*) and moments be taken about A, the moment will be

$$M = H.S \tag{88}$$

But if the bunker be assumed as an equilibrium polygon drawn by using a force polygon, the bending moment at the center is equal to the pole distance multiplied by the intercept S. Therefore H must be equal to the pole distance of the force polygon.

Now it has been proved (The Design of Steel Mill Buildings, Chapter XVa) that the differential equation of the elastic curve is

$$\frac{d^2y}{dx^2} = \frac{fx}{H} \tag{89}$$

where fx is the loading and H is the pole distance of the force polygon as shown in (*d*) Fig. 78.

But

$$fx = p = \frac{l - x}{l} P, \text{ and}$$

$$\frac{d^2y}{dx^2} = \frac{P}{H} - \frac{P.x}{l.H} \text{ is the differential equation of the curve.}$$

Integrating we get

$$\frac{dy}{dx} = \frac{P.x}{H} - \frac{P.x^2}{2l.H} + C_1 \tag{90}$$

Now $\frac{dy}{dx} = 0$ when $x = 0$, and $C_1 = 0$.

Integrating equation (90)

$$y=\frac{P.x^2}{2H}-\frac{P.x^3}{6l.H}+C_2 \tag{91}$$

Now $y=0$ when $x=0$, and $C_2=0$.

Therefore we have

$$y=\frac{P.x^2}{2H}-\frac{P.x^3}{6l.H} \tag{92}$$

When $x=l$, then $y=S$, and

$$S=\frac{P.l^2}{3H}$$

and

$$H=\frac{P.l^2}{3S} \tag{93}$$

Substituting the value of H given in (93) in (92), we have

$$y=\tfrac{1}{2}\frac{S}{l^2}\left(3x^2-\frac{x^3}{l}\right) \tag{94}$$

which is the final equation of the curve of the bunker. This equation shows that the shape of the curve is independent of the maximum load and depends only on the width and sag. The capacity of the bunker is as shown in (a) equal to

$$C=2l.S-2a.$$

But

$$a=\int_0^l y.dx=\tfrac{3}{2}\,\frac{S}{l^2}\int_0^l x^2.dx-\frac{S}{2l^3}\int_0^l x^3.dx$$

$$=\tfrac{3}{8}l.S,$$

and

$$C=2l.S-2.\tfrac{3}{8}l.S$$

$$=\tfrac{5}{4}l.S \tag{95}$$

In calculating the value of P for any given bunker, since P is the maximum pressure for a triangular loading, we have

$$P.\,l=C.w, \text{ and}$$

$$P=\frac{C.w}{l} \tag{96}$$

where w = weight of a unit of the filling. For a bunker level full

$$P=\tfrac{5}{4}S.w$$

The curve may be constructed graphically as follows: In (*c*), Fig. 78, it is required to pass the curve through the points *A* and *B*. The loads 1, 2, 3, 4, etc., are laid off in the force polygon (*d*), and a pole *O* is taken. The equilibrium polygon *A-B′* is then constructed in (*c*). Now we know from graphic statics that if two poles be taken for the force polygon in (*d*), and corresponding equilibrium polygons be drawn through *A*, the strings meeting on the same load will intersect on a line through *A* parallel to the line *O-O′*. Now *D* is determined by the intersection of rays *D-B′* and *D-B*. The true curve is then easily constructed and pole *O′* is located.

If the bunker is surcharged by vertical walls as shown in (*e*) the curve is extended until it meets the slope of the material, and the span and sag are to be used as shown.

Diagram for Suspension Bunker.—In Fig. 79 is given a diagram proposed by R. W. Dull, for calculating the stresses, capacity, length of curve, etc., of suspended bunkers. The bunker selected is one that approximates an equilateral triangle. The use of the diagram will be explained by means of an example.

Problem 1. It is required to design a 400-ton bunker 100 feet long. This makes the capacity 4 tons per foot of length. Start on the right side at the 4-ton mark and pass to the left until the area curves are intersected and the width and sag will be found on the lower side. We see that if the bin is to be surcharged at an angle of 35° the width will be 17 feet, with a sag of 10.2 feet, while if the bin is to be level the width will be nearly 21 feet, with a sag of 12.6 feet. Assuming the surcharged bin with a width of 17 feet, we find the length of the curve on the upper side to be 340 inches. Passing over to the left on the 4-ton line we find the area of the section 160 square feet; reaction $V=4{,}000$ lbs.; passing on to the intersection of the 4,000-lb. line with the diagonal line we find the stress per foot of length of plate, $T=4{,}600$ lbs., and the tension $H=2{,}300$ lbs. This furnishes the necessary information for the design.

Diagrams for other shapes of bunkers may be constructed in the same manner.

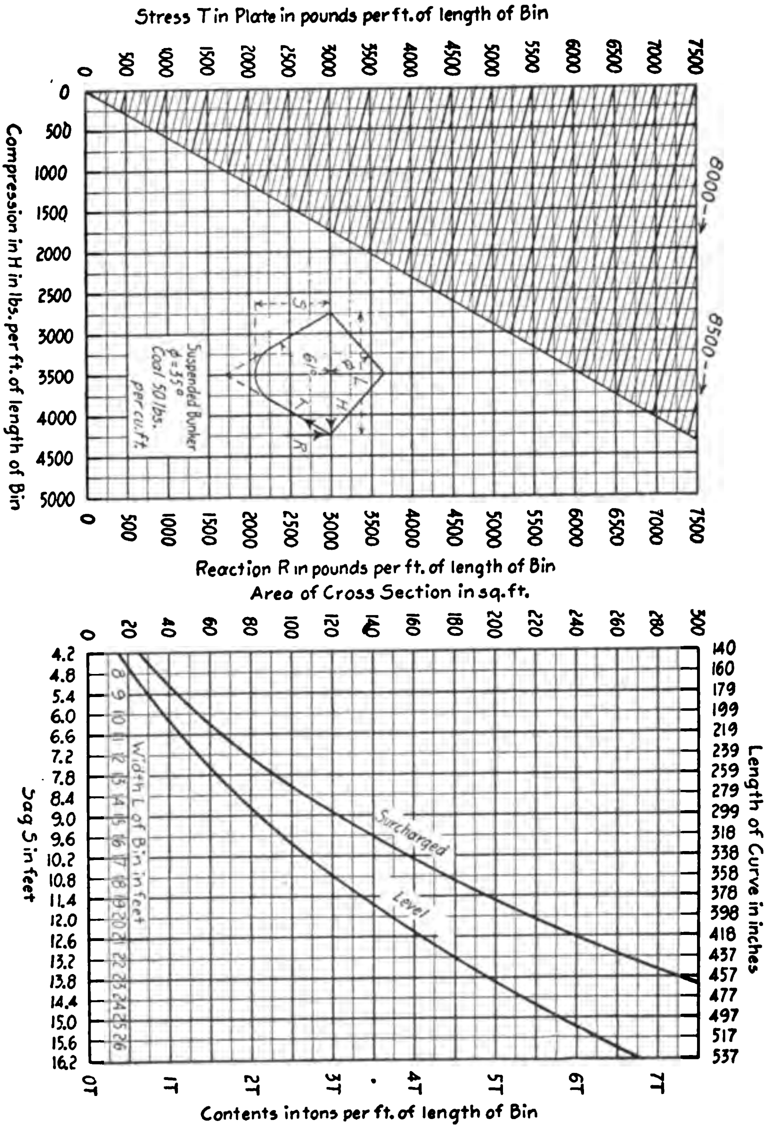


FIG. 79. DIAGRAM FOR DESIGNING A SUSPENSION BUNKER.

4. STRESSES IN CONICAL BIN BOTTOMS.—A conical bin with a radius r is supported by a circular girder, carries a filling with a weight w and angle of repose ϕ. It is required to calculate the stresses in the conical bottom and in the girder. If the sides of the bin are smooth or the depth of the bin is small the entire weight will be carried on the bottom of the bin. If the bin is deep the vertical pressure, V, on the bottom of the conical part may be found by equation (144). The top surface of the material will be assumed to be level.

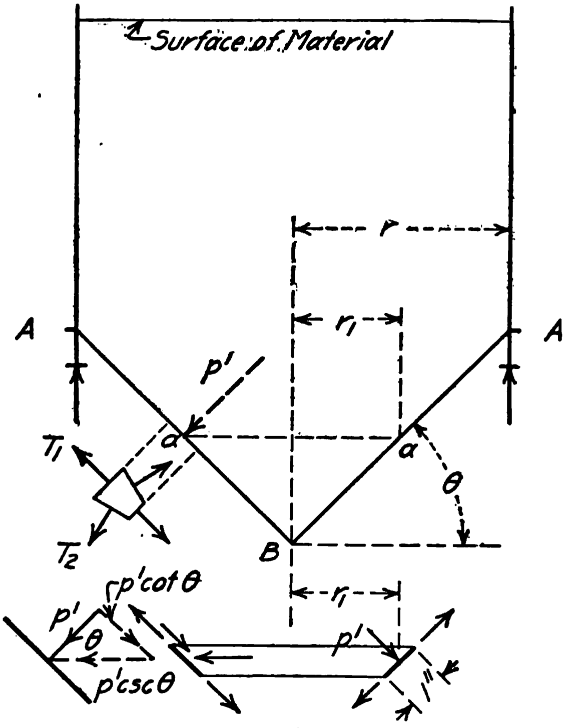


Fig. 80.

Let V be the vertical pressure at any point in the granular mass, then the horizontal component will be $L = V\dfrac{1 - \sin\phi}{1 + \sin\phi}$*. Any horizontal joint in the cone must carry a load W_1 equal to the total vertical pressure on the circle cut by the section a-a plus the weight of the cone and filling below the plane of *a-a*. To calculate W_1 in a deep bin substitute in formula (144) using r_1 for the radius of the bin and $\phi' = \phi$, because the filling tends to slide on itself.

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* More correctly $L = k,V$, where k is determined by experiment, see Chapter XVIII.

Now the stress in the cone will be tangential to the shell and

$$T_1 = \frac{W_1 \,.\, csc\,\theta}{2\pi . r_1} = \tfrac{1}{2} V . r_1 \,.\, csc\,\theta \tag{97}$$

which varies from zero at B to a maximum at A.

Shallow Bins.—To find the stress in the ring *a-a*, assume a section of unit slant height cut from the cone. Now the normal pressure p' on the element may be found from the ellipse of stress and is given by the formula

$$p' = V \frac{\sin^2(\theta + \phi)}{\sin^3\theta\left(1 + \frac{\sin\phi}{\sin\theta}\right)^2}$$

Now substitute for the pressure p' which acts normally to the ring the two components $p' \,.\, \cot\theta$ which acts along an element, and $p' \,.\, \csc\theta$ which causes tension in the ring. Then

$$T_2 = p' \,.\, r_1 \,.\, csc\,\theta \tag{98}$$

Stress T_2 varies from zero at B to a maximum at A.

Deep Bins.—It may easily be proved that

$$T_2 \doteq L \,.\, r_1 \tag{98a}$$

The circular girder which carries the entire weight of the bin and contents is carried at three or more points, so that it is subjected to compression, bending, and torsion. The compression in the ring is $\frac{W \,.\, \cot\theta}{2\pi}$ where W is the total weight of bin and filling. The bending and torsion are calculated as described under stresses in a circular girder.

5. **STRESSES IN SPHERICAL BIN BOTTOMS.**—The stress T_1 is calculated in the same manner as for the conical bin and

$$T_1 = \frac{W_1}{2\pi . r' \sin^2 \alpha'} = \tfrac{1}{2} V . r' \tag{99}$$

The tension on any meridian joint *a-a* is

$$T_2 = \tfrac{1}{2}V.r' = T_1 \qquad (100)$$

where V is calculated by formula (144) for the cylinder *a-a* as for the conical bin. The stresses in a spherical bottom are less than in a conical bottom.

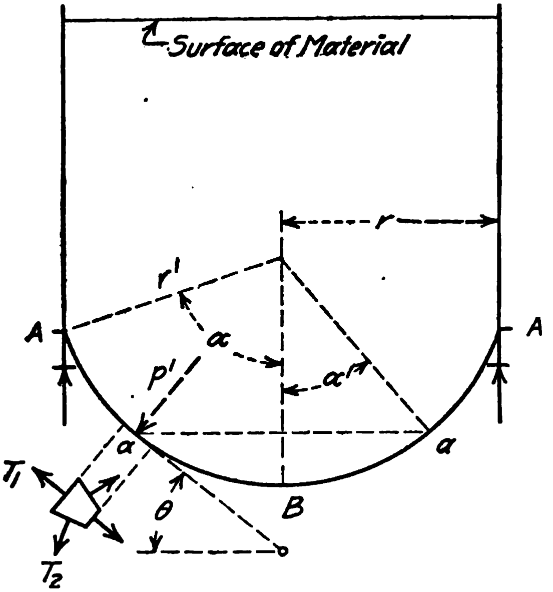


FIG. 81.

6. STRESSES IN A CIRCULAR GIRDER.

—The circular girder supports the weight of the bin, the contents of the bin, and its own weight. The load is uniformly distributed along the girder. The girder rests on or is supported by four or more columns, and transmits its load to them.

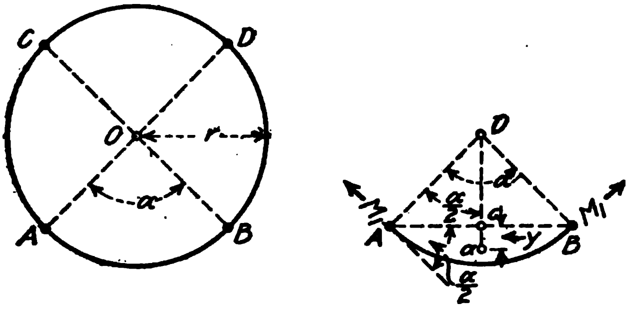
Let W = total load on girder in pounds,

r = radius of girder in inches,

n = number of posts,

$a = \frac{2\pi}{n}$ = angle at center subtended by radii through two consecutive posts,

a' = angle subtended at center by any arc,
M = direct bending moment in the girder at any point in in.-lbs.,
T = torsional bending moment in girder at any point in in.-lbs.,
S = shear in girder at any point,
$P_a = P_b$, etc., = reactions of posts.



(a) (b)

FIG. 82.

Now cut away the girder as in (*b*), Fig. 82, and replace the stresses and moments in the parts cut away by external forces and moments. The shear to the left of A is $S=\frac{W}{2n}$. The section of girder A-B tends to drop downward like a cantilever beam producing a moment $= -M_1$ at each post tangent to the circle at A and B. If a is the center of gravity of arc A-B, and y the distance of a from the line A-B, then

$$-2M_1=\frac{W}{n}\,y.csc\frac{a}{2},\text{ but}$$

$$Oa=\frac{2r.\sin\frac{a}{2}}{a},\text{ and}$$

$$y=2r\left(\frac{\sin\frac{a}{2}}{a}-\tfrac{1}{2}\cos\frac{a}{2}\right). \qquad (101)$$

and substituting the value of y, we have

$$-M_1=\frac{W.r}{n}\left(\frac{\sin\frac{\alpha}{2}}{\alpha}-\tfrac{1}{2}\cos\frac{\alpha}{2}\right)csc\,\frac{\alpha}{2}$$

$$=\frac{W.r}{n}\left(\frac{1}{\alpha}-\tfrac{1}{2}\cot\frac{\alpha}{2}\right) \qquad (102)$$

If $n=4$, then $\alpha=\frac{\pi}{2}$, and

$$M_1=\frac{W.r}{2}\left(\tfrac{1}{4}-\frac{1}{\pi}\right) \qquad (103)$$

$$=-0.03415\,W.r$$

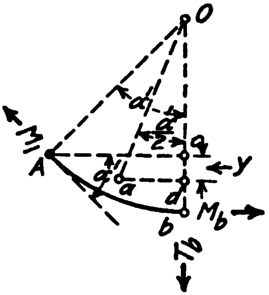


FIG. 83.

At any point in the girder, Fig. 83, we will have a direct moment M_b tangent to the girder and a torsional moment T_b normal to the girder. Taking moments about *b-O*, we have

$$M_b=M_1\,.\cos\alpha'+(P_a-S)cA-\frac{W.\alpha'.ad}{2\pi} \qquad (104)$$

Now

$$cA=r.\sin\alpha'$$

and

$$Oa=\frac{2r.\sin\frac{\alpha'}{2}}{\alpha'},\text{ and } ad=\frac{2r.\sin^2\frac{\alpha'}{2}}{\alpha'},$$

$$M_b=M_1\cos\alpha'+\frac{W}{2n}\,r.\sin\alpha'-\frac{W.r.\sin^2\frac{\alpha'}{2}}{\pi} \qquad (105)$$

When

$$\alpha'=0$$

$$M_b=M_1$$

and when $\alpha' = \frac{\alpha}{2}$, we have bending moment at a point midway between posts $= M_2$

$$M_2 = M_1 . \cos\frac{\alpha}{2} + \frac{W.r}{2n}\left(\sin\frac{\alpha}{2} - \frac{2\sin^2\frac{\alpha}{4}}{\frac{\alpha}{2}}\right) \qquad (106)$$

If $n = 4$, then $\alpha = \frac{\pi}{2}$, and

$$M_2 = M_1 . \cos 45^\circ + \frac{W.r}{8}\left(\sin 45^\circ - \frac{8\sin^2 22\frac{1}{2}^\circ}{\pi}\right), \text{ and}$$

$$M_2 = 0.01762\, W.r$$

To find the point of contra-flexure place $M_b = 0$, and solve for α'. Assume $n = 4$, and

$$M_b = \frac{W.r}{2}\left(\frac{1}{4} - \frac{1}{\pi}\right)\cos\alpha' + \frac{W.r}{8}\sin\alpha' - \frac{W.r}{\pi}\sin^2\frac{\alpha'}{2} = 0 \qquad (107)$$

which after reduction gives

$$\cos\alpha' + \sin\alpha' = \frac{4}{\pi} \qquad (108)$$

and

$$\alpha' = 19^\circ\ 12' \text{ or } 70^\circ\ 48'$$

The torsional moment at any point b is found by taking moments about a tangent at b.

$$T_b = -M_1 \sin\alpha' - (P_a - S)\, cb + \frac{W.\alpha'}{2\pi}\, db \qquad (109)$$

Now

$$cb = r(1 - \cos\alpha'),$$

$$db = r - oa.\cos\frac{\alpha'}{2}$$

$$= r - \frac{2r.\sin\frac{\alpha'}{2}\cos\frac{\alpha'}{2}}{\alpha'}$$

$$= r\left(1 - \frac{\sin\alpha'}{\alpha'}\right)$$

$$T_b = -M_1.\sin\alpha' - \frac{W}{2n}\, r(1 - \cos\alpha') + \frac{W.\alpha'}{2\pi}\, r\left(1 - \frac{\sin\alpha'}{\alpha'}\right) \qquad (110)$$

Now when $a' = 0$, then $T_1 = 0$.

When $a' = \frac{a}{2}$, then

$$T_b = -M_1 . \sin\frac{a}{2} - \frac{W.r}{2n}\left(1 - \cos\frac{a}{2}\right) + \frac{W.a.r}{4\pi}\left(1 - \frac{\sin\frac{a}{2}}{a}\right) = 0 \quad (111)$$

To find the maximum value of T_b differentiate (110) with reference to a'

$$dT_b = \left(-M_1.\cos a' - \frac{W.r}{2n}\sin a' + \frac{W.r}{2\pi} - \frac{W.r\cos a'}{2\pi}\right)da' = 0 \quad (112)$$

and

$$-M_1.\cos a' - \frac{W.r}{2n}\sin a' + \frac{W.r}{2\pi}(1 - \cos a') = 0 \quad (113)$$

If $n = 4$, then

$$M_1 = \frac{W.r}{2}\left(\tfrac{1}{4} - \frac{1}{\pi}\right),$$

and

$$\cos a' + \sin a' = \frac{4}{\pi}, \quad (114)$$

and

$$a' = 19^\circ\ 12', \text{ or } 70^\circ\ 48'$$

and

$$T_b = 0.0053\, W.r$$

From the preceding discussion we see that the bending moment M_1 has maximum values at the posts and midway between the posts and is zero at the points of contra-flexure; while the torsional moment T_1 has maximum values at the points of contra-flexure, and is zero at points of maximum bending moments.

TABLE XXIVa.

Stresses in Circular Girder.*

No. of Posts.	Load on Post, Lbs.	Max. Shear, Lbs.	Bending Moment at Posts. In.-lbs.	Bending Moment Midway Between Posts. In.-lbs.	Angular Distance From Post to Point of Max. Torsion.	Max. Torsional Moment. In.-lbs.
4	$W \div 4$	$W \div 8$	$-0.03415\, Wr$	$+0.01762\, Wr$	19° 12′	$0.0053\, Wr$
6	$W \div 6$	$W \div 12$	$-0.01482\, Wr$	$+0.00751\, Wr$	12 44	$0.00151\, Wr$
8	$W \div 8$	$W \div 16$	$-0.00827\, Wr$	$+0.00416\, Wr$	9 33	$0.00063\, Wr$
12	$W \div 12$	$W \div 24$	$-0.00365\, Wr$	$+0.00190\, Wr$	6 21	$0.000185\, Wr$

* From article by H. J. Burt, M. Am. Soc. C. E., in Technograph No. 16.

The shear, bending moments and torsional moments calculated by the above formulas for circular girders having 4, 6, 8, and 12 posts are given in Table XXIVa.

7. **STRESSES IN COLUMNS.**—The stresses in the columns will be due to the dead load and to the wind moment. The dead load stress will be equal to W divided by the number of columns, where W is the total weight of bin and filling. To calculate the stresses due to wind moment in the columns proceed as follows: Calculate the wind force by multiplying the exposed surface by the wind pressure, and assume the wind force as acting through the center of gravity of the exposed surface. The pressure on circular bins may be taken at from 18 to 30 lbs. per sq. ft. of surface at right angles to the direction of the wind, depending upon the location and surroundings. To calculate the stresses in the columns at any point pass a horizontal section through the columns as in Fig. 84. Then the maximum vertical stress in column 1 will occur on the leeward side when the wind is blowing in the direction 1–1. If M is the wind moment about the axis A-B, the moment of the stresses in the column about axis A-B will be equal to M. In a bin with 8 columns as in Fig. 84 we have stress $1 \times 2r +$ stress $2 \times 4r.\cos 45° = M$.

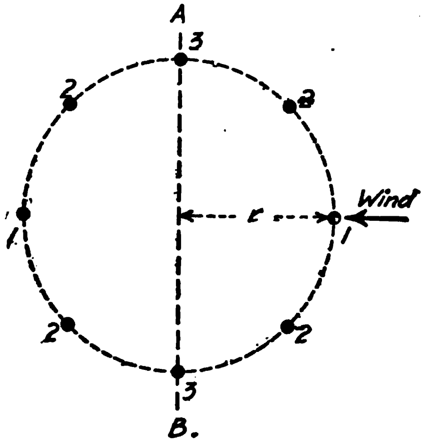


FIG. 84.

But stress 1 is to stress 2 as r is to $r.\cos 45°$. From which

$$\text{Stress } 1\,(2r+2r)=M$$

$$\text{Stress } 1=\frac{M}{4r}, \text{ and}$$

$$\text{Stress } 2=\frac{0.7M}{4r}$$

In a 6 column bin the stress in the most remote post is $\frac{M}{3r}$, and on each of the others is $\frac{1}{2}\frac{M}{3r}$. In a 4 column bin the stress in each post is $\frac{M}{2r}$. If the columns are vertical the maximum stresses will occur at the foot of the columns; if the columns are inclined the stress should be calculated at both the top and the bottom. The maximum stresses will be the sum of the dead and wind load stresses.

If the upward pull of the columns on the windward side is greater than the dead load when the bin is empty the column must be anchored down. The masonry footing should have a weight equal to at least twice the resultant upward pull.

For the calculation of footings see "Steel Mill Buildings," Chapter XXI.

CHAPTER X.

EXPERIMENTS ON PRESSURE ON BIN WALLS.

Introduction.—Comparatively few experiments have been made on the pressure of coal, ore, ashes, etc., on bin walls. The following are all that have been available to the author:

Portland Cement Materials.* Experiments were made by L. D. Gilbert and S. F. Barth at the Portland Cement Co.'s plant, Portland, Colorado, in January, 1906, to determine data to use in the design of cement bins. The results are given in Table XXV. In making the tests great care was used, each value given being the mean of three independent values.

The materials were weighed in a box holding one cubic foot, the material was packed by tapping the sides and jarring the bottom and was then struck level, the aim being to obtain as nearly as possible the same conditions that exist in a stock bin. The fineness was determined by standard sieves.

In making the tests for lateral pressure a small bin was built of dressed Mexican clear lumber, three feet square inside, and four feet deep, with an opening 12 inches square cut through one side starting at the bottom, the hole being cut with a bevel to allow ample clearance in the opening. A door was hinged on the bottom of the opening with steel butts. The door was hung with the center line of butts flush with the inside face and set inside of bin a scant $\frac{1}{8}$ inch. The opening was covered with a light piece of linen duck stretched tight and tacked all around about three or four inches from the hole, forming a diaphragm. An iron bracket was screwed on the outside of the door and the pressure was carried to the scale platform. The scale used was a small platform scale weighing to one-half pound.

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* Courtesy of Mr. L. D. Gilbert.

Before putting any material in the bin the scale was tested with a standard 50-lb. weight, and the weight of the lever and door noted. Material to the depth of 12 inches was then put in the bin and the resultant pressure on the scales noted, then another 12 inches of material was added and the pressure noted, and so on. In each case the surface was leveled and brought to the proper height.

In making the three tests on the same material the pressure ran very close, and as the angle of repose decreased the lateral pressure increased.

In all cases, especially with the finer materials, the pressure was greatest at the time the bin was filled, and fell off considerably upon standing an hour or so, indicating that in a large bin requiring a considerable time to fill the pressure would be less than in a small bin.

TABLE XXV.

EXPERIMENTS ON PORTLAND CEMENT MATERIALS.

Material.	Weight per Cu. Ft. in Pounds.	Per Cent Passing.	Angle of Repose ϕ.	Ratio of Vertical to Lateral Press.
Clinker.	85.3	All passed 1½″ ring.	33°	.30
Cement from Kom's.	97.0	95% through No. 20.	37.5°	.32
Finished cement.	103.0	96% through No. 100.	See Fig. 85 a	.42
Rock from crusher (Shale).	89.7	All passed 2½″ ring.	39°	.28
Raw mat. from Kom's.	79.9	90% through No. 20.	33°	.40
Raw mat. from tube mills.	67.6	89% through No. 100.	See Fig. 85 b	.42
Coal-slack.	53.0	All passed 2″ ring.	37.5°	.31
Coal from Williams mill.	50.6	63% through No. 10.	34.5°	.33
Coal from tube mills.	42.0	98% through No. 100.	16°	.41

The Portland cement assumed a curved surface of repose as shown in Fig. 85 a; while the material from the tube mills assumed the curved surface shown in Fig. 85 b.

Coal and Ashes.—The author can find no records of tests for powdered coal aside from those given by Gilbert and Barth. The values for weight and angle of repose in use by different firms and engineers are given in Table XXVI. It should be remembered that the coarser the material the less the pressure.

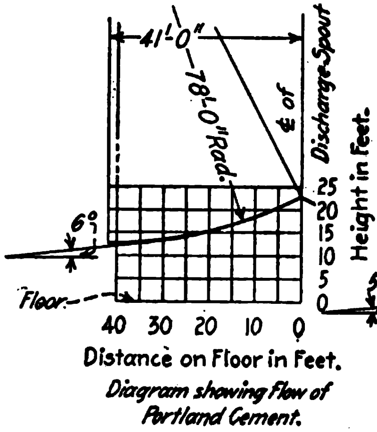

FIG. 85 a.

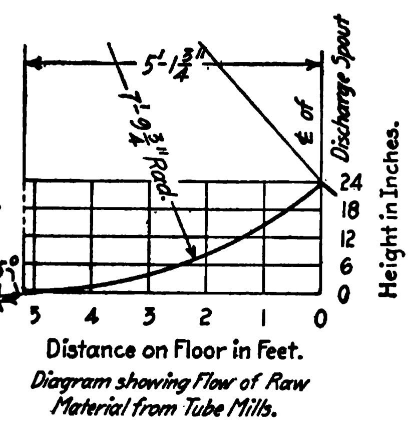
5'-1¾"
7'-9¾" Rad.
℄ of Discharge Spout
5½°
24
18
12
6
0
Height in Inches.
5 4 3 2 1 0
Distance on Floor in Feet.
Diagram showing Flow of Raw Material from Tube Mills.

FIG. 85 b.

TABLE XXVI.

WEIGHT AND ANGLE OF REPOSE OF COAL, COKE, ASHES AND ORE.

Material.	Weight Lbs. per Cu. Ft.	Angle of Repose ϕ in Degrees.	Authority.
Bituminous coal.	50	35	Link Belt Machinery Co.
Bituminous coal.	47	35	Link Belt Engineering Co.
Bituminous coal.	47 to 56		Cambria Steel.
Anthracite coal.	52	27	Link Belt Machinery Co.
Anthracite coal.	52.1	27	Link Belt Engineering Co.
Anthracite coal fine.		27	K. A. Muellenhoff.
Anthracite coal.	52 to 56		Cambria Steel.
Slaked coal.		45	Wellman-Seaver-Morgan Co.
Slaked coal.	53	37½	Gilbert and Barth.
Coke.	23 to 32		Cambria Steel.
Ashes.	40	40	Link Belt Machinery Co.
Ashes, soft coal.	40 to 45		Cambria Steel.
Ore, soft iron.		35	Wellman-Seaver-Morgan Co.

Mr. J. E. A. Moore, engineer, The Wellman-Seaver-Morgan Company, states that coal, ore, etc., in lumps will commonly slide at an angle of 30° if the material is dry, but if the material is wet the angle of repose may increase to nearly 90°. For dry slack coal he uses $\phi = 45°$ and for soft iron ore, comparatively dry, he uses $\phi = 35°$.

Angle of Friction on Bin Walls.—The values differ with materials and with the bin wall. The following values may be used in the absence of more accurate data:

TABLE XXVII.

ANGLE OF FRICTION OF DIFFERENT MATERIALS ON BIN WALLS.

Material.	Steel Plate. ϕ' in Degrees.	Wood Cribbed. ϕ' in Degrees.	Concrete. ϕ' in Degrees.
Bituminous coal.	18	35	35
Anthracite coal.	16	25	27
Ashes.	31	40	40
Coke.	25	40	40
Sand.	18	30	30

Self-Cleaning Hoppers.—In order to have hoppers self-cleaning when the material is moist it is necessary, especially with fine material, to have the hopper bottoms slope at an angle considerably in excess of the angle of repose ϕ' or ϕ.

Ore pockets on the Great Lakes are made with hopper bottoms having an angle with the horizontal of from 38° 40′ to 50° 45′, but the majority are at an angle of 39° to 45°.

Crushed quartz from 1½ in. to ⅛ in. in size was found to readily slide down a steel plate floor at an angle of 40°, while ore finer than ⅛ in. would not move. Experiments showed that bituminous coal would slide down a steel chute at an angle of 40°, and down a wooden chute at 45° to the horizontal. Anthracite will slide down steel at 30° and down wood at 35°.*

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* Richard Lamb, in Tran. Am. Soc. C. E., Vol. 53.

CHAPTER XI.

THE DESIGN OF BINS.

Introduction.—The problem of the design of the structural part of steel bins is similar to the design of Steel Mill Buildings, and the reader is referred to the author's book on the Design of Steel Mill Buildings. For the design of reinforced concrete bins the reader is referred to Chapter IV and Chapter XVIII, also to the descriptions of actual structures in Chapters XII and XIX. The design of circular bins is taken up in Chapter XIX. In this chapter only the details of design that are fundamental or are peculiar to bins are treated.

Flat Plates.—The analysis of the stresses in flat plates supported or fixed at their edges is extremely difficult. The following formulas by Grashof may be used: The coefficient of lateral contraction is taken as ¼. For a full discussion of these formulas based on Grashof's "Theorie Der Elasticitat und Festigkeit" see Lanza's Applied Mechanics.

1. *Circular plate of radius r and thickness t, supported around its perimeter and loaded with w per square inch.*—Let f = maximum fiber stress, v = maximum deflection, and E = modulus of elasticity.

$$f=\frac{117}{228}\,\frac{w.r^2}{t^2} \tag{115}$$

$$v=\frac{189}{256}\,\frac{w.r^4}{E.t^3} \tag{116}$$

2. *Circular plate built in or fixed at the perimeter.*

$$f=\frac{45}{64}\,\frac{w.r^2}{t^2} \tag{117}$$

$$v=\frac{45}{256}\,\frac{w.r^4}{E.t^3} \tag{118}$$

3. *Rectangular plate of length a, breadth b, and thickness t, built in or fixed at the edges and carrying a uniform load w per square inch.*—

Let f_a be the unit stress parallel to a, f_b be the unit stress parallel to b, *and* $a > b$.

$$f_a = \frac{b^4.w.a^2}{2(a^4+b^4)t^2}\,;\ f_b = \frac{a^4.w.b^2}{2(a^4+b^4)t^2} \tag{119}$$

$$v = \frac{a^4.b^4.w}{(a^4+b^4)32\,E.t^3} \tag{120}$$

For a square plate $a=b$,

$$f = \frac{w.a^2}{4t^2} \tag{121}$$

$$v = \frac{w.a^4}{64E.t^3} \tag{122}$$

The strength of plates simply supported on the edges is about ⅔ the strength of plates fixed. Plates riveted or bolted around the edges may be considered as fixed.

Diagram for Square Plates.—The safe loads on square plates for a fiber stress of 10,000 pounds per square inch may be obtained from the diagram in Fig. 86. As an example, required the safe load for a ¼-in. plate 3 feet square. Begin at 3 on the bottom of the diagram, follow upward to the line marked ¼-in. plate, from the intersection follow to the left edge and find 280 lbs. per sq. ft. For any other fiber stress multiply the safe load found from the diagram by the ratio of the fiber stresses. To use the diagram for a rectangular plate take a square plate having the same area.

Buckle Plates.—Buckle plates are made by "dishing" flat plates as in Table XXVIIIa. The width of the buckle W or length L, varies from 2′ 6″ to 5′ 6″. The buckles may be turned with the greater dimension in either direction of the plate. Several buckles may be put in one plate, all of which must be the same size and symmetrically placed. Buckle plates are made ¼″, 5/16″, ⅜″ and 7/16″ in thickness. The common standard sizes are given in Table XXVIIIa.

Buckle plates should be firmly bolted or riveted around the edges with a maximum spacing of 6 inches, and should be supported transversely between the buckles. The process of buckling distorts the

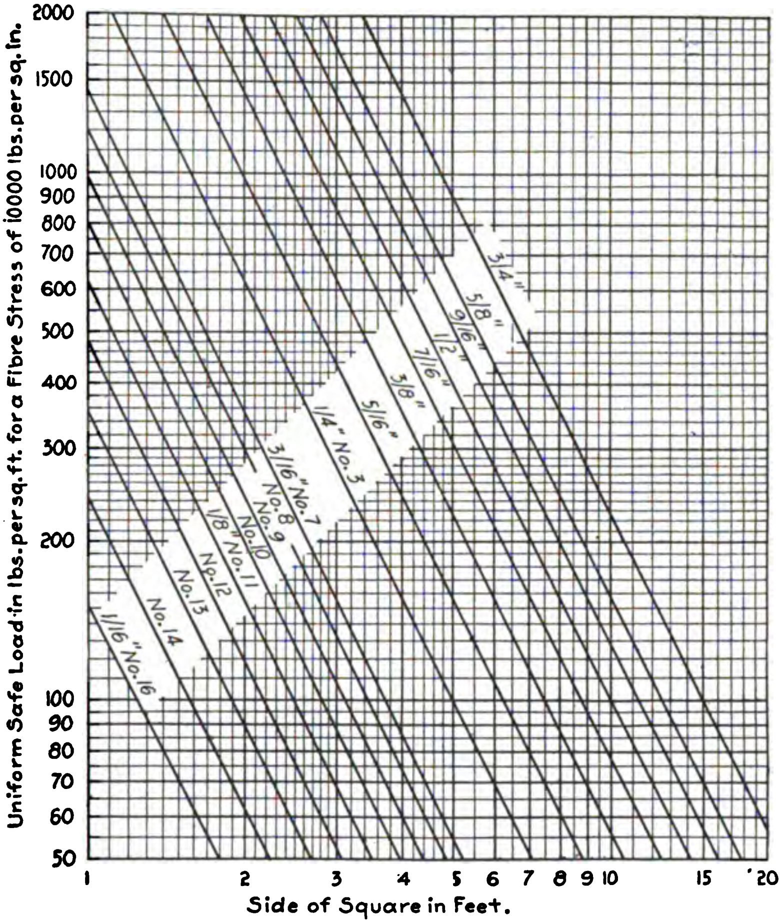


FIG. 86. SAFE UNIFORM LOAD PER SQUARE FOOT ON FLAT PLATES FOR A FIBER STRESS OF 10,000 LBS. PER SQUARE INCH.

plate and an extra width should be ordered and the plate should be trimmed after the process is complete.

Strength of Buckle Plates.—The safe load for a buckle plate with buckles placed up, is approximately given by the formula

$$W = 4f.R.t \tag{123}$$

where W = total safe uniform load,

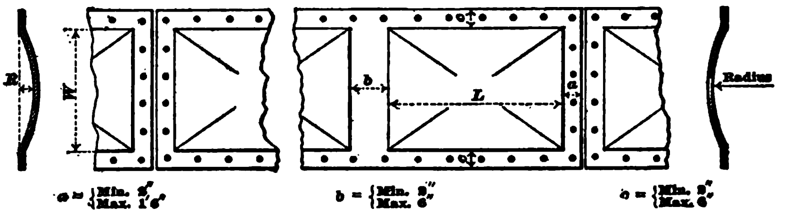
f = safe unit stress in pounds per square inch,

R = depth of buckle in inches,
t = thickness of plate in inches.

Where buckle plates are riveted and the buckle placed down the safe load is from 3 to 4 times that given above.

TABLE XXVIIIa.

AMERICAN BRIDGE CO.'S STANDARD BUCKLE PLATES.



No of Plate.	Size of Buckle in Feet and Inches.		Rise in Inches.	Rad. of Buckle in Feet and Inches.		Maximum No. of Buckles.	No. of Plate.	Size of Buckle in Feet and Inches.		Rise in Inches.	Rad. of Buckle in Feet and Inches.		Maximum No. of Buckles.
	Length. L	Width W	R	Length. L	Width. W			Length. L	Width. W	R	Length. L	Width. W	
1	3-11	4-6	3½	6-8	8-9	7	20	2-9	2-6	2½	4-7	3-10	10
2	4-6	3-11	3½	8-9	6-8	6	21	2-6	2-6	2½	3-10	3-10	10
3	3-11	3-6	3	7-9	6-3	7	22	3-5	3-6	3	5-11	6-3	8
4	3-6	3-11	3	6-3	7-9	8	23	3-6	3-5	3	6-3	5-11	8
5	3-9	3-9	3	7-1	7-1	8	24	3-6	3-9	3	6-3	7-1	8
6	3-1	3-9	3	4-10	7-1	9	25	3-9	3-6	3	7-1	6-3	8
7	3-9	3-1	3	7-1	4-10	8	26	3-1	3-2	3	4-10	5-1	9
8	3-8	3-8	2	10-2	10-2	8	27	3-2	3-1	3	5-1	4-10	9
9	2-8	3-8	2	5-5	10-2	10	28	3-1	3-0	3	4-10	4-7	9
10	3-8	2-8	2	10-2	5-5	8	29	3-0	3-1	3	4-7	4-10	9
11	2-2	3-8	2	3-7	10-2	10	30	2-0	2-6	2½	2-6	3-10	10
12	3-8	2-2	2	10-2	3-7	8	31	2-6	2-0	2½	3-10	2-6	15
13	3-0	3-0	2	6-10	6-10	9	32	3-6	5-6	3½	5-4	13-1	5
14	2-9	2-9	3	3-10	3-10	10	33	5-6	3-6	3½	13-1	5-4	1
19	2-6	2 9	2½	3-10	4-7	10	34	4-0	4-0	3	8-1	8-1	7

Plates are made $\frac{1}{4}''$, $\frac{5}{16}''$, $\frac{3}{8}''$ or $\frac{7}{16}''$ thick.
Buckles of different sizes should not be used in the same plate.
Rivets generally $\frac{5}{8}''$ or $\frac{3}{4}''$ diameter.

Rivets and Riveted Joints.—Plates are fastened together with rivets in several different ways. If the plates are lapped over each other the joint is a *lap* joint, the joint being called a *single lap,* Fig. 87 a; *double lap,* Fig. 87 b; *triple lap,* etc., depending upon the number of rows of rivets. If the plates are fastened together with one splice plate as in Fig. 88 a, the joint is called a *single strap butt* joint; while if two plates are used, as in Fig. 88 b, the joint is called a *double strap butt* joint. Butt joints may have single, double, etc., riveting on each side of the joint. Where more than one row of rivets is used on one side of a joint the rivets in one row are placed opposite the middle of the rivets in the other row, *i. e.*, the rivets are *staggered.* When joints are designed to be water tight the edges of the plates are beveled and the joint is calked after riveting by driving the metal of the beveled edge back into the joint with a chisel, called a *calking tool.*

Investigation of Riveted Joints.—*Lap joint single riveting.* In Fig. 87 a let P be the tensile stress transmitted from one plate to another by a single rivet, t be the thickness of the plate, p be the pitch of the rivets, and d be the diameter of a rivet. Let S_t be the tensile unit-stress which occurs in the plate between the rivets, S_s the unit shearing stress in the rivet, and S_c the unit compression on the vertical projection of the rivet at right angles to the line of stress.

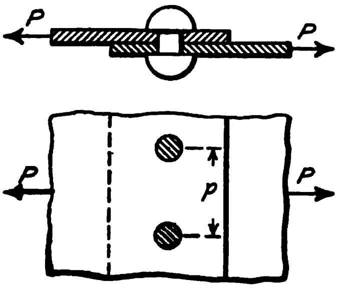


FIG. 87 a.

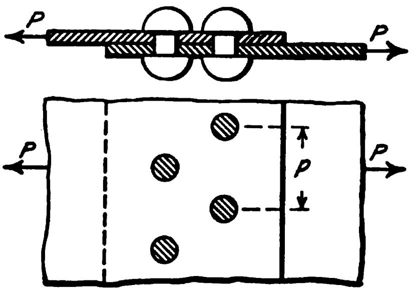


FIG. 87 b.

Then the equations between the stresses and force P are

$$t(p-d)S_t = P \text{ for tension on plate,} \tag{124}$$

$$t.d.S_c = P \text{ for compression on rivet,} \tag{125}$$

$$\tfrac{1}{4}\pi.d^2.S_s = P \text{ for shear on rivet.} \tag{126}$$

For a lap joint with double riveting, let P be the tension on a width p. Then the equations between the stresses and the force P are

$$t(p-d)S_t=P \tag{127}$$

$$2t.d.S_c=P \tag{128}$$

$$2\frac{\pi}{4}.d^2.S_s=P \tag{129}$$

For a butt joint with a single strap and a single row of rivets, as in Fig. 88 a, the joint becomes two single riveted lap joints and the same equations may be used. For a butt joint with double strap plates and a single row of rivets on each side the following equations are used in designing the joint, t being the thickness of the main plate:

$$t(p-d)S_t=P \tag{130}$$

$$t.d.S_c=P \tag{131}$$

$$2\frac{\pi}{4}.d^2.S_s=P \tag{132}$$

The strap plates should never be less than one-half the thickness of the main plate.

While for a butt joint with double strap plates and double riveting on each side, as in Fig. 88 b, the following equations are used in designing the joint, t being the thickness of the main plate:

$$t(p-d)S_t=P \tag{133}$$

$$2t.d.S_c=P \tag{134}$$

$$4\frac{\pi}{4}.d^2.S_s=P \tag{135}$$

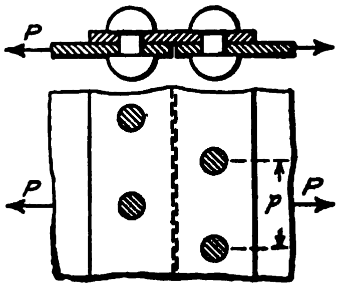


Fig. 88 a.

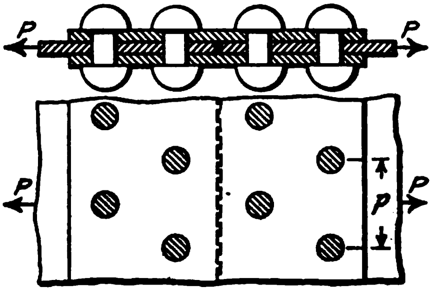


Fig. 88 b.

The investigation of a joint consists in calculating the values of S_t, S_c, and S_s, and comparing them with the allowable values. The allowable values for S_t, S_c, and S_s are not usually the same. In the author's specifications for Steel Frame Buildings as given in the Design of Steel Mill Buildings, the following allowable stresses are given: $S_t = 16{,}000$ lbs., $S_c = 22{,}000$, and $S_s = 11{,}000$ lbs. per sq. in.

The "efficiency" of a joint is the ratio of its allowable stress to the allowable stress on the unriveted plate. For a riveted joint three efficiencies may be obtained by dividing the three values of P for tension, compression and shear by $p.t.S_t$, the strength of the unriveted plate. For example, let $S_t = 16{,}000$, $S_c = 22{,}000$, and $S_s = 11{,}000$ lbs. per sq. in., and let $p = 2\frac{1}{2}''$, $d = \frac{7}{8}''$, and $t = \frac{3}{8}''$, for a lap joint with double riveting. Then the efficiency for tension is $(p - d)t.S_t/p.t.S_t = \frac{(p-d)}{p} = 0.65$; the efficiency for compression is $2t.d.S_c/p.t.S_t = 2d.S_c/p.S_t = 0.96$; and the efficiency for shear is $\frac{1}{2}\pi.d^2.S_s/p.t.S_t = 0.88$. The efficiency of this joint is therefore 65 per cent of the unriveted plate.

For bins subjected to severe usage the usual values for the allowable stresses are $S_t = 10{,}000$, $S_c = 15{,}000$, and $S_s = 7{,}500$ lbs. per sq. in. For these values the efficiencies of the above joint are, tension $= 0.65$, compression $= 1.00$, and shear $= 0.96$. By equating the three expressions for efficiency, values of t, d, and p can be calculated. Using $S_t = 10{,}000$, $S_c = 15{,}000$, and $S_s = 7{,}500$ lbs. per sq. in., and solving as above we have for a riveted lap joint with single riveting

$$d = 2.55\,t$$

$$p = 6.37\,t$$

$$\text{Efficiency} = 0.60$$

For a lap joint with double riveting

$$d = 2.55\,t$$

$$p = 10.2\,t$$

TABLE XXIX.

Shearing and Bearing Value of Rivets.

Values Above or to Right of Upper Zigzag Lines are Greater Than Double Shear. Values Below or to Left of Lower Zigzag Lines are Less Than Single Shear.

Diam. of Rivet.		Area in Sq. Ins	Single Shear at 6,000 Pounds	Bearing Value for Different Thickness of Plate at 12,000 Pounds per Square Inch.												
Frac.	Dec'l.			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{3}{8}$	.375	.1104	660	1,130	1,410	1,690										
$\frac{1}{2}$	.500	.1963	1,180	1,500	1,880	2,250	2,630	3,000								
$\frac{5}{8}$	.625	.3068	1,840	1,880	2,340	2,810	3,280	3,750	4,220	4,690						
$\frac{3}{4}$	.750	.4418	2,650	2,250	2,810	3,380	3,940	4,500	5,160	5,630	6,190	6,750				
$\frac{7}{8}$	.875	.6013	3,610	2,630	3,280	3,940	4,590	5,250	5,910	6,560	7,220	7,880	8,530	9,190	9,840	
1	1.000	.7854	4,710	3,000	3,750	4,500	5,250	6,000	6,750	7,500	8,250	9,000	9,750	10,500	11,250	12,000

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 7,500 Pounds	Bearing Value for Different Thickness of Plate at 15,000 Pounds per Square Inch.												
Frac.	Dec'l.			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{3}{8}$	.375	.1104	830	1,410	1,760	2,110										
$\frac{1}{2}$	.500	.1963	1,470	1,880	2,340	2,810	3,280	3,750								
$\frac{5}{8}$	.625	.3068	2,300	2,340	2,930	3,520	4,100	4,690	5,280	5,860						
$\frac{3}{4}$	.750	.4418	3,310	2,810	3,520	4,220	4,920	5,630	6,330	7,030	7,720	8,440				
$\frac{7}{8}$	.875	.6013	4,510	3,280	4,100	4,920	5,740	6,560	7,380	8,200	9,030	9,850	10,670	11,480	12,300	
1	1.000	.7854	5,890	3,750	4,690	5,620	6,560	7,500	8,440	9,380	10,310	11,250	12,190	13,130	14,060	15,000

TABLE XXIX.—*Continued.*

SHEARING AND BEARING VALUE OF RIVETS.

Values Above or to Right of Upper Zigzag Lines are Greater Than Double Shear. Values Below or to Left of Lower Zigzag Lines are Less Than Single Shear.

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 11,000 Pounds	Bearing Value for Different Thickness of Plate at 22,000 Pounds per Square Inch.												
Frac.	Dec'l.			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	1,210	2,060	2,580	3,090										
1/2	.500	.1963	2,160	2,750	3,440	4,130	4,820	5,500								
5/8	.625	.3068	3,370	3,440	4,300	5,160	6,020	6,880	7,740	8,600						
3/4	.750	.4418	4,860	4,130	5,160	6,190	7,220	8,250	9,280	10,320	11,340	12,380				
7/8	.875	.6013	6,610	4,810	6,020	7,220	8,430	9,630	10,840	12,040	13,240	14,440	15,640	16,840	18,050	
1	1.000	.7854	8,640	5,500	6,880	8,250	9,630	11,000	12,380	13,750	15,130	16,500	17,880	19,520	20,630	22,000

Diam. of Rivet.		Area in Sq. Ins.	Single Shear at 12,000 Pounds.	Bearing Value for Different Thickness of Plate at 24,000 Pounds per Square Inch.												
Frac.	Dec'l.			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
3/8	.375	.1104	1,320	2,250	2,810	3,380										
1/2	.500	.1963	2,360	3,000	3,750	4,500	5,250	6,000								
5/8	.625	.3068	3,680	3,750	4,690	5,620	6,560	7,500	8,440	9,370						
3/4	.750	.4418	5,300	4,500	5,620	6,750	7,870	9,000	10,120	11,250	12,370	13,500				
7/8	.875	.6013	7,220	5,250	6,560	7,870	9,190	10,500	11,810	13,120	14,440	15,750	17,060	18,370		
1	1.000	.7854	9,430	6,000	7,500	9,000	10,500	12,000	13,500	15,000	16,500	18,000	19,500	21,000	22,500	24,000

$$\text{Efficiency} = 0.75$$

This shows the advantage of double riveting.

Where the rivet holes in the plates are made by punching specifications require that the diameter of the rivet before driving be used in calculating the strength of the rivet, and that the rivet hole be taken $\frac{1}{8}$ inch larger than the undriven rivet.

To obtain a tight joint the rivets should be spaced close together and as near the edge as practicable to make it possible to thoroughly calk the joint; while for strength the rivets should be spaced far apart and placed well back from the edge.

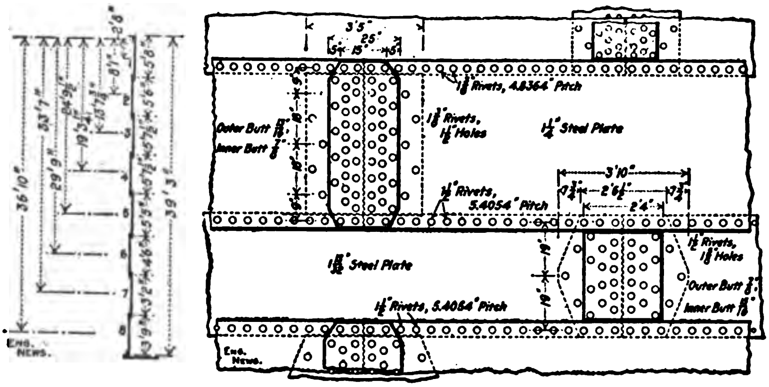
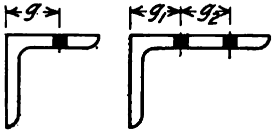


FIG. 89. RIVETING ON GAS HOLDER.

The arrangement of plates and the rivet details for the vertical butt joints and horizontal lap joints for a gas holder are shown in Fig. 89.

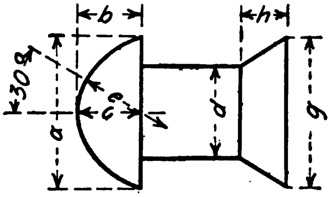


GAGES
in inches

Leg	Gage g	Max. Rivet	Leg	Gage g_1	Gage g_2	Max. Rivet
8	$4\frac{1}{2}$	$\frac{7}{8}$	8	3	3	$\frac{7}{8}$
7	4	$\frac{7}{8}$	7	$2\frac{1}{2}$	3	$\frac{7}{8}$
6	$3\frac{1}{2}$	$\frac{7}{8}$	6	$2\frac{1}{4}$	$2\frac{1}{4}$	$\frac{7}{8}$
5	3	$\frac{7}{8}$	5	2	$1\frac{3}{4}$	$\frac{7}{8}$
4	$2\frac{1}{2}$	$\frac{7}{8}$	When 6" L exceeds $\frac{3}{4}$"			
$3\frac{1}{2}$	2	$\frac{7}{8}$	6	$2\frac{1}{2}$	2	$\frac{7}{8}$
3	$1\frac{3}{4}$	$\frac{7}{8}$				
$2\frac{3}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$				
$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$				
$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$				
2	$1\frac{1}{8}$	$\frac{5}{8}$				
$1\frac{3}{4}$	1	$\frac{1}{2}$				
$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$				
$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{3}{8}$				
$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{3}{8}$				
1	$\frac{5}{8}$	$\frac{1}{4}$				

MINIMUM RIVET SPACING

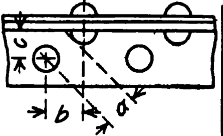
Size of Rivet inches	Min. Distance inches
1	3
$\frac{7}{8}$	$2\frac{5}{8}$
$\frac{3}{4}$	$2\frac{1}{4}$
$\frac{5}{8}$	2
$\frac{1}{2}$	$1\frac{3}{4}$
$\frac{3}{8}$	$1\frac{1}{4}$
$\frac{1}{4}$	1



PROPORTIONS OF RIVETS
in inches

Diameter of Shank d	Full Head				Countersunk	
	Diameter a	Height b	Radii c	Radii e	Diameter g	Depth h
1	$1\frac{5}{8}$	$\frac{11}{16}$	$\frac{11}{16}$	$1\frac{1}{32}$	$1\frac{9}{16}$	$\frac{1}{2}$
$\frac{7}{8}$	$1\frac{7}{16}$	$\frac{39}{64}$	$\frac{39}{64}$	$\frac{59}{64}$	$1\frac{3}{8}$	$\frac{7}{16}$
$\frac{3}{4}$	$1\frac{1}{4}$	$\frac{17}{32}$	$\frac{17}{32}$	$\frac{51}{64}$	$1\frac{3}{16}$	$\frac{3}{8}$
$\frac{5}{8}$	$1\frac{1}{16}$	$\frac{29}{64}$	$\frac{29}{64}$	$\frac{43}{64}$	1	$\frac{5}{16}$
$\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{9}{16}$	$\frac{25}{32}$	$\frac{1}{4}$
$\frac{3}{8}$	$\frac{11}{16}$	$\frac{19}{64}$	$\frac{19}{64}$	$\frac{7}{16}$	$\frac{19}{32}$	$\frac{3}{16}$

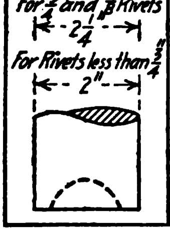
MINIMUM STAGGER FOR RIVETS



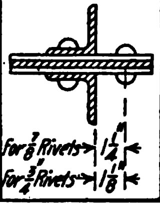
c inches	b in inches: For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$"	b in inches: For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$"
$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
$1\frac{3}{16}$	$1\frac{3}{16}$	$1\frac{7}{16}$
$1\frac{1}{4}$	$1\frac{1}{8}$	$1\frac{3}{8}$

c inches	b in inches: For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$"	b in inches: For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$"
$1\frac{5}{16}$	$1\frac{1}{16}$	$1\frac{5}{16}$
$1\frac{3}{8}$	$\frac{15}{16}$	$1\frac{1}{4}$
$1\frac{7}{16}$	$\frac{7}{8}$	$1\frac{3}{16}$
$1\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{8}$
$1\frac{9}{16}$	$\frac{9}{16}$	1
$1\frac{5}{8}$	$\frac{3}{8}$	$\frac{15}{16}$
$1\frac{11}{16}$	0	$\frac{13}{16}$
$1\frac{3}{4}$		$\frac{5}{8}$
$1\frac{13}{16}$		$\frac{7}{16}$
$1\frac{7}{8}$		0

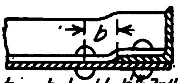
MINIMUM BUTTON SETS


For $\frac{3}{4}$" and $\frac{7}{8}$" Rivets
$2\frac{1}{4}$
For Rivets less than $\frac{3}{4}$"
2"

STANDARD CLEARANCE FOR RIVETING



RIVETS IN CRIMPED Ls



Distance b should = $1\frac{1}{2}$" + 2 × thickness chord Ls, but never less than 2"

FIG. 90. AMERICAN BRIDGE CO.'S STANDARDS FOR RIVETS AND RIVETING.

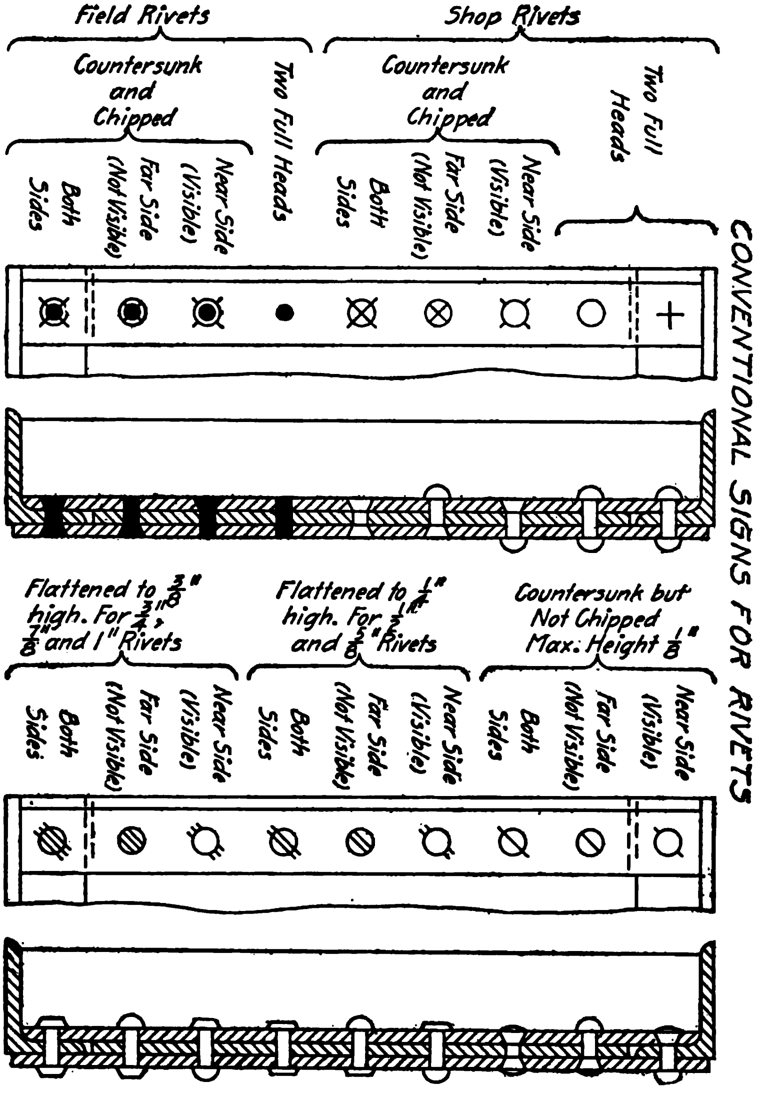


FIG. 91. CONVENTIONAL SIGNS FOR RIVETS.

The shearing and bearing value of rivets for different unit stresses are given in Table XXIX.

The proportions of rivets and standards for riveting structural steel as adopted by the American Bridge Company are given in Fig. 90.

In structural drawing it is necessary to use conventional methods to indicate the different kinds of riveting. The conventional signs for riveting in use by the American Bridge Co. are given in Fig. 91.

The standard rivet spacing for calking as used by the Gillette-Herzog Manufacturing Co. is given in Table XXX.

TABLE XXX.

STANDARD SPACING AND EDGE DISTANCE IN INCHES OF RIVETED JOINTS FOR CALKING.

Thickness of Plate.	⅜" Rivets.			½" Rivets.			⅝" Rivets.			¾" Rivets.			⅞" Rivets.		
	p	p'	E	p	p'	E	p	p'	E	p	p'	E	p	p'	E
⅛"	1¼	2	⅝												
3/16	1½	2¼	¾	1½	2¼	¾									
¼	1¼	2¼	¾	1¾	2½	⅞	1⅞	2⅝	1	2	2¾	1⅛			
5/16				1¾	2½	⅞	2	2¾	1	2⅛	2⅞	1⅛			
⅜				1⅞	2⅝	1	2	2¾	1	2¼	3	1⅛	2⅜	3⅛	1¼
7/16							2⅛	2⅞	1	2¼	3	1⅛	2⅜	3¼	1⅜
½							2¼	3	1⅛	2⅜	3⅛	1¼	2½	3¼	1½
⅝										2½	3¼	1¼	2⅝	3⅜	1½

p = pitch for single riveting, p' = pitch for double riveting, and E = edge distance.

Design of Stiffeners.—Stiffeners are riveted to the sides of bin plates to carry part of the flexural stress and also to transmit the shear as in plate girders. In designing the stiffener to take flexural stress it is usual to assume that the stiffener acts as a beam and carries the load on one-half the area on either side. This makes the stiffener a beam carrying a wedge-shaped mass as in Fig. 92.

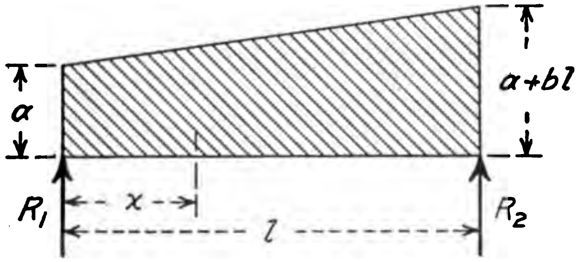


FIG. 92.

$$R_1 = \frac{a.l}{2} + \tfrac{1}{6}b.l$$

$$R_2 = \frac{a.l}{2} + \tfrac{1}{3} b.l$$

The bending moment at any point x, is

$$M = R_1.x - \left(\frac{a.x^2}{2} + \frac{b.x^3}{6}\right) \qquad (136)$$

Solving for a maximum

$$dM = R_1.dx - a.x.dx - \frac{b.x^2.dx}{2} = 0, \text{ and}$$

$$x = -\frac{a}{b} + \sqrt{\frac{a^2}{b^2} + \frac{a.l}{b} + \tfrac{1}{3} l} \qquad (137)$$

and the maximum bending moment will be found by substituting the value of x in (136).

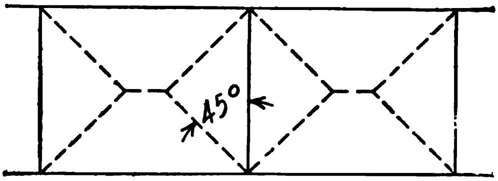


FIG. 93.

Where the stiffener supports an area as in Fig. 93 the loading will be unsymmetrical, and can most easily be found by graphics. The stiffener may be assumed to carry the load as indicated in Fig. 93 by the dotted lines.

Bin Gates.—Many gates have been devised to make it possible to handle the different materials quickly and easily. The gates in Fig. 94 have been designed by the Link Belt Machinery Co., and are especially adapted to coal and similar materials. Gate (f) is an ordinary slide gate, while the other gates are of the under-cut type.

Gate (a) in Fig. 95 operates very easily, while (b) is the type of gravity gate commonly used in ore bins. A detailed description of gate (b) is given on page 183. Gate (c) is often used in grain bins.

The gate in Fig. 96 is of the under-cut type and was used on an ore bin.

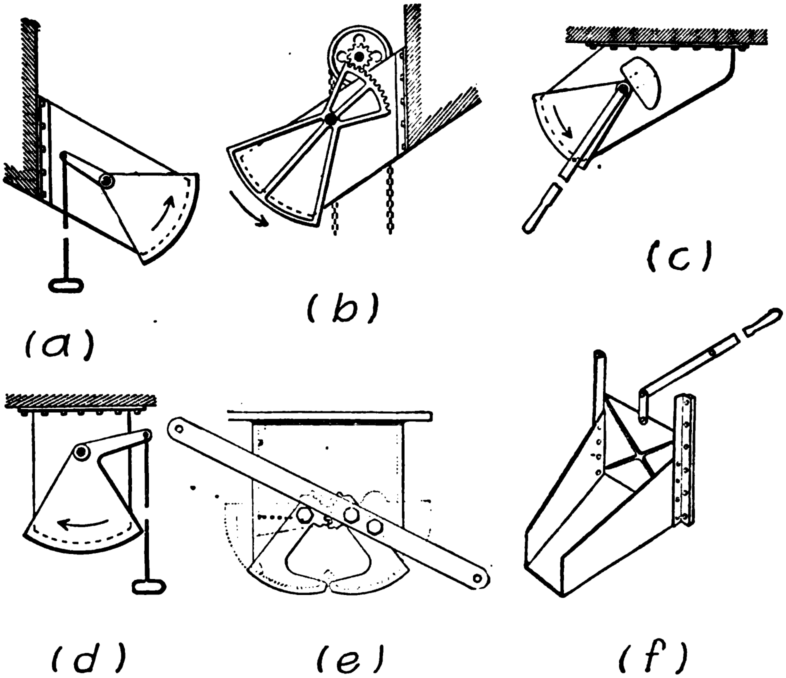


FIG. 94.

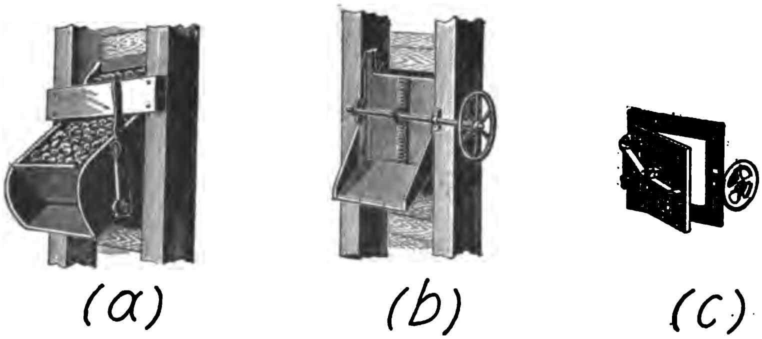


FIG. 95.

The gate shown in Fig. 97 a was designed by the Link Belt Engineering Co., and was used on the bin in Fig. 105.

The gate shown in Fig. 97 b was used on the circular ore bins shown in Fig. 58.

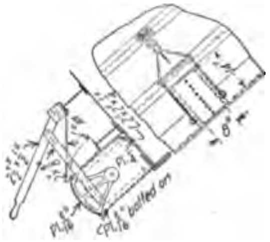
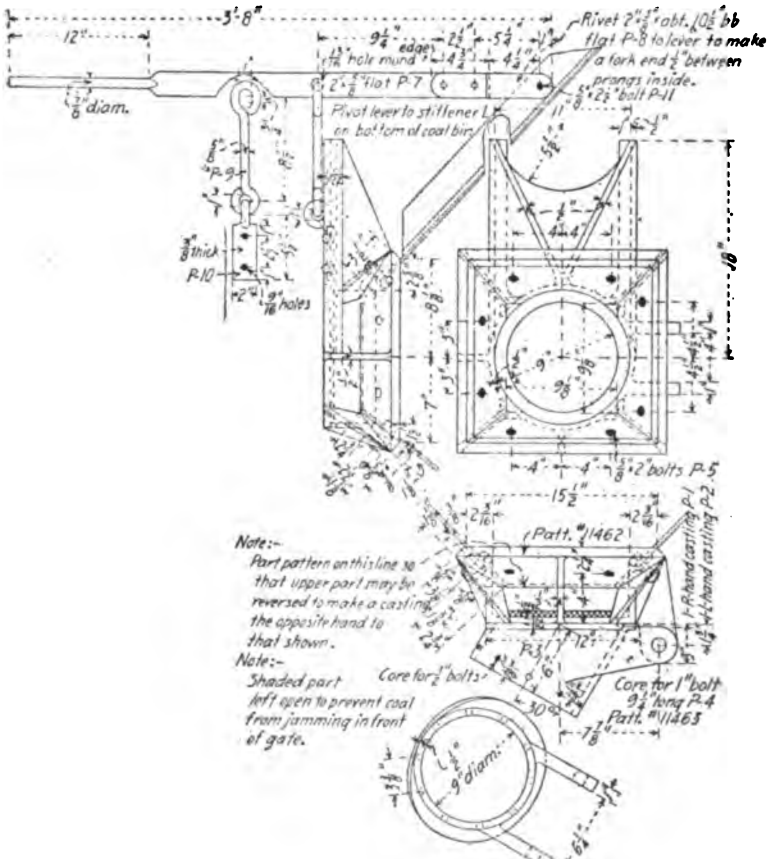


FIG. 96.


3'-8"
12"
⅞" diam.
9¼"
edge
2½"
5¼"
½"
13/16" hole mund
4¾"
4¼"
2"×⅝" flat P-7
Rivet 2"×⅝" abt. 10½" lg flat P-8 to lever to make a fork end ½" between prongs inside.
⅝"×2½" bolt P-11
Pivot lever to stiffener L on bottom of coal bin
11"
5⅝"
P-9
⅜" thick
P-10
2"
9/16" holes
18"
9"
9⅝"
4"
4"
⅝"×2" bolts P-5
15½"
2 3/16"
2 3/16"
Patt. #11462
1-R-hand casting P-1
1-L-hand casting P-2
Note:-
Part pattern on this line so that upper part may be reversed to make a casting the opposite hand to that shown.
Note:-
Shaded part left open to prevent coal from jamming in front of gate.
P-3
12"
Core for ½" bolts
30°
Core for 1" bolt 9¼" long P-4 Patt. #11463
7⅞"
9" diam.

Hinged Gate Casting—Left hand as shown.

FIG. 97 a.

For additional details of gates see Fig. 56, Fig. 98, Fig. 111, and Fig. 117.

Specifications.—The allowable stresses in bin plates and the structural framework supporting the bins will depend upon the use that is to be made of the bin. The factor of safety should vary from 6 for quiescent loads to 4 for bins that are to be subjected to hard usage. All steel should be made by the open hearth process, structural or tank steel plates being used for flat plates and flange steel being used for all bent or flanged plates.

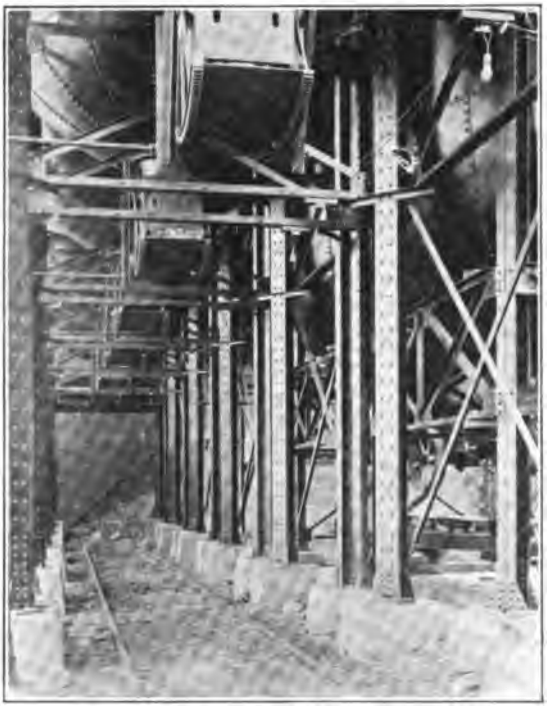


Fig. 97 b. Gate for Circular Ore Bin, Arizona. (See Bin in Fig. 58 and Fig. 118.)

The specifications for materials and workmanship for steel structures adopted by the American Railway Engineering and Maintenance of Way Association, and the American Society for Testing Materials are standard and are given in Appendix II. For ordinary conditions in bin design use the allowable stresses given, while for bins to receive severe treatment use ⅔ of the allowable stresses.

For specifications for Steel Frame Mill Buildings, see the author's specifications in "The Design of Steel Mill Buildings."

Painting.—Paints used on structural work may be divided into oil paints, asphalt paints, coal tar paints, etc. The oil paints are made by grinding or mixing a pigment with linseed oil. The common pigments are red lead, white lead, oxide of iron, zinc and graphite. For a description of the different kinds of oil paints see Chapter XXVII in the author's "The Design of Steel Mill Buildings."

Asphalt paint is made by mixing asphalt with a bitumen, the mixture being applied hot. Many so-called asphalt paints are only coal tar mixtures.

Coal tar paint is occasionally used for painting gas tanks and similar structures that receive rough usage. Coal tar paint mixed as described below has been used by the U. S. Navy Department for painting the hulls of ships. It should give satisfactory service where the metal is subject to corrosion. The coal tar paint is mixed as follows: The proportions of the mixture are slightly variable according to the original consistency of the tar, the use for which it is intended and the climate in which it is used. The proportions will vary between the following proportions in volume.

	Coal Tar.	Portland Cement.	Kerosene Oil.
New Orleans Mixture..................	8	1	1
Annapolis Mixture	16	4	3

The Portland cement should first be stirred into the kerosene, forming a creamy mixture, the mixture is then stirred into the coal tar. The paint should be freshly mixed and kept well stirred. This paint sticks well, does not run when exposed to the sun's rays and is a very satisfactory paint for rough work. The cost of the paint will vary from 10 to 20 cents per gallon. The kerosene oil acts as a dryer, while the Portland cement neutralizes the coal tar.

Carbon and graphite paints are much used for painting structural and bin work. While the cost per gallon of these paints is usually rather high, $1.00 to $1.50 per gallon, the paints cover so much surface that the actual cost of the paint is not high.

The Wellman-Seaver-Morgan Co. recommends American graphite mixed with boiled linseed oil for painting the structural framework and outside of the bin plates. This company recommends that steel bins be lined with a layer of concrete about 1½" thick, the concrete being reinforced with expanded metal or wire fabric. This concrete lining has been found to protect the surface of the steel both from wear and corrosion.

The American Bridge Co. uses linseed oil mixed with ¾ lb. of lampblack per gallon as a priming coat with very satisfactory results. For further information on paint see "The Design of Steel Mill Buildings," Chapter XXVII. For the cost of painting see Chapter XIII.

CHAPTER XII.

EXAMPLES AND DETAILS OF BINS.

Introduction.—Examples of bins built for different purposes will be given in this chapter, and details of the different structures will be briefly discussed. The examples have been grouped under the following heads: (1) Suspension Bunkers; (2) Hopper Bins; (3) Circular Bins. For a brief description of these different types see Chapter VIII.

1. SUSPENSION BUNKERS. Lackawanna Steel Co.'s Plant.—In Fig. 98 and Fig. 99 are shown the details of a panel of the ore and limestone bins at the Lackawanna Steel Co.'s plant, Buffalo, N. Y., built by Wellman-Seaver-Morgan Company, Cleveland, Ohio. The bin is of the suspension bunker type with $\frac{3}{8}''$ hopper plates. The tracks on the top of the bin are for the traveling gantry bins and dump cars. The material is discharged from the bins through spouts onto larrys running on the track beneath the spouts. The hopper plates are $\frac{3}{8}''$ thick. It will be seen in Fig. 99 that openings are provided in the hopper just above the spouts to enable the material inside the bin to be loosened up. The details and construction are fully shown in Fig. 98 and Fig. 99.

The details of a 1,500 ton coal bin built by the Wellman-Seaver-Morgan Company for the above company are shown in Fig. 100. This bin is of the suspension type with a double discharge spout, under which is a Seaver ore gate. The coal stored in this bin is crushed and is quite wet. It is delivered into the top of the bin by a belt conveyor. The 12″ × 14″ hole in the spout is to allow bars to be inserted to loosen the material in the bins. Details of the bin are fully shown in the drawing.

The bin is lined with a concrete lining reinforced with expanded metal and finished with a coating of one to one Portland cement mortar. The bin plates are ⅜″ thick. The bin is covered with corrugated steel supported on a steel framework. The entire structure was painted

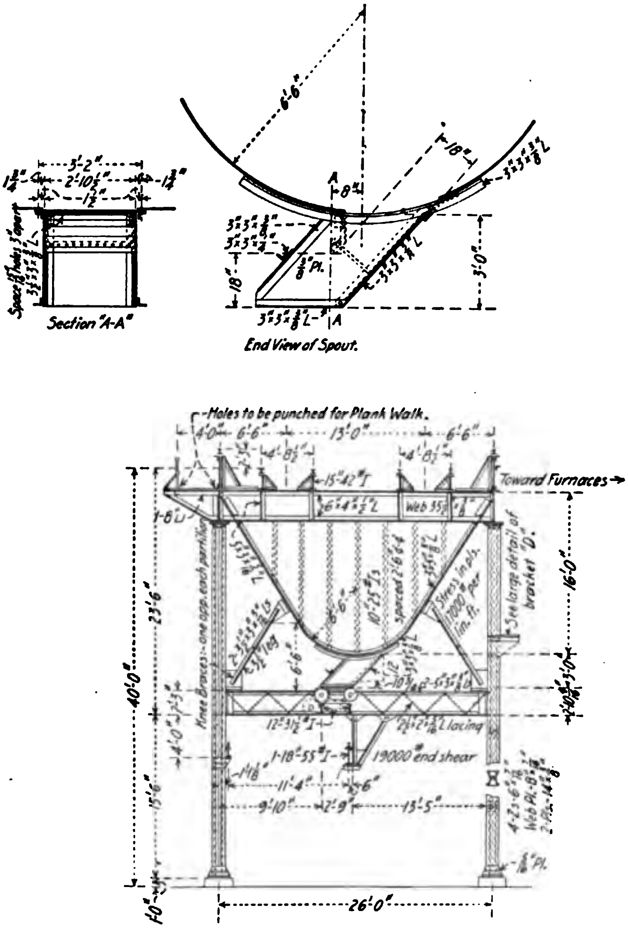


FIG. 98. ORE AND LIMESTONE BUNKERS, LACKAWANNA STEEL CO.

one coat of graphite paint in the shop and one coat after erection. The load was taken as 23 tons per lineal foot of bin. The stresses in the framework and the plates and other details are given in Fig. 100.

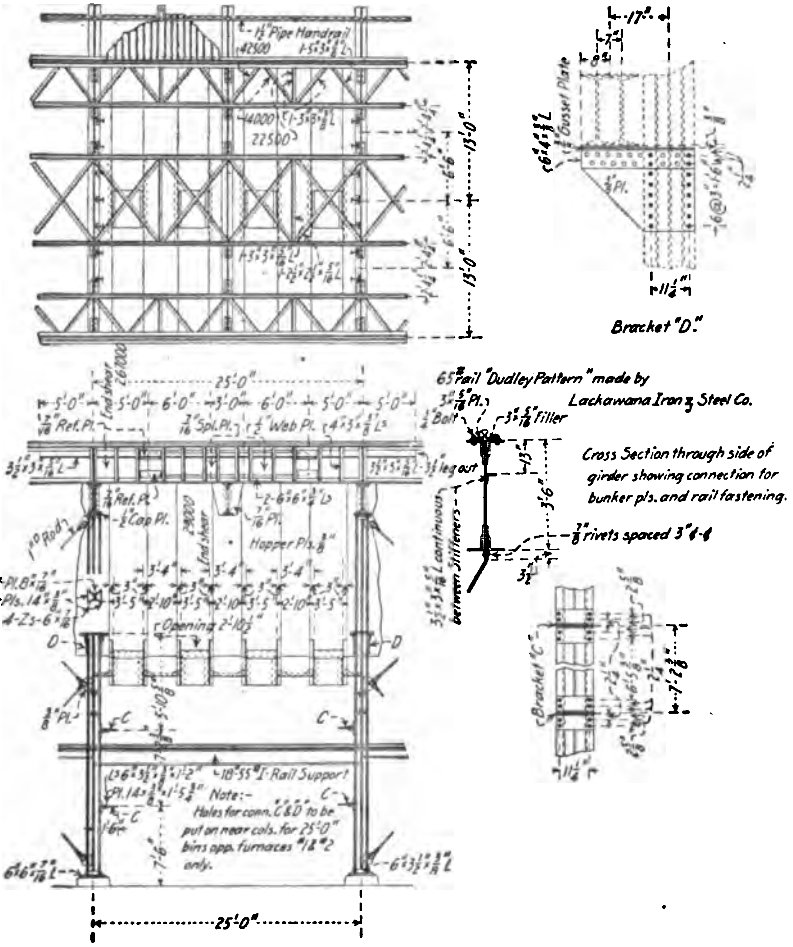
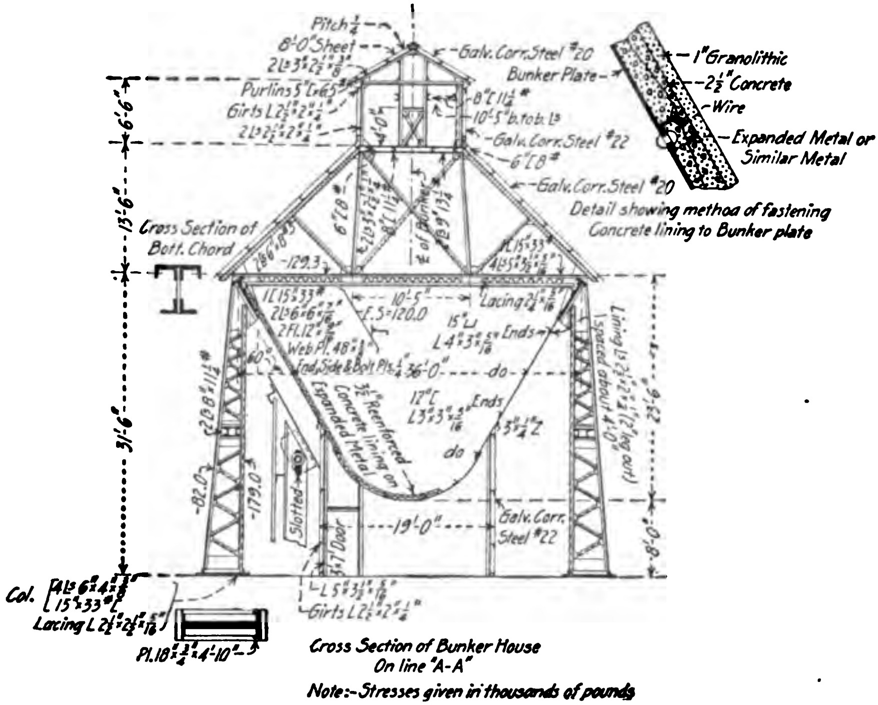

1½" Pipe Handrail
13'-0"
6'-6"
25'-0"
5'-0"
6'-0"
3'-0"
Ref. Pl.
Spl. Pl.
Web Pl.
Cap Pl.
Hopper Pls.
End shear
Opening 2'-10½"
D
C
18" 55# I-Rail Support
Note:- Holes for conn. C & D to be put on near cols. for 25'-0" bins opp. furnaces #1 & 2 only.
25'-0"
Bracket "D."
Gusset Plate
65# rail "Dudley Pattern" made by Lackawanna Iron & Steel Co.
Filler
Bolt
Cross Section through side of girder showing connection for bunker pls. and rail fastening.
rivets spaced 3"4-4
continuous between Stiffeners
Bracket "C"

FIG. 99. ORE AND LIMESTONE BUNKERS, LACKAWANNA STEEL CO.

Rapid Transit Subway Power House Coal Bunkers.—The 1,000 ton suspension coal bunker at the Fifty-eighth St. and Eleventh Ave. power house of the Rapid Transit Subway is shown in Fig. 101 and Fig. 102.

The bunker is supported on braced posts and is covered with a steel frame house carrying the Robins belt conveyor. The bin is lined with 3½ inches of concrete reinforced with expanded metal. The loads and

stresses in frame work and plates are given in Fig. 101. The bunker has 2½″ × 2″ × ¼″ angles placed horizontally 4′ 0″ apart and riveted to the bunker plates. The bunker is covered with galvanized corrugated steel supported on a steel framework. The details of construction are clearly shown in the drawings. This bunker was designed and constructed by the American Bridge Co.



Cross Section of Bunker House
On line "A-A"

Note:—Stresses given in thousands of pounds

FIG. 101. COAL BUNKERS, RAPID TRANSIT SUBWAY, NEW YORK, N. Y.

2. **HOPPER BINS. Cananea Consolidated Copper Co.**—The hopper ore bin shown in Fig. 103 was built by the Minneapolis Steel & Machinery Co. for the Cananea Consolidated Copper Co., Cananea, Mexico.

The ore is coarse and heavy and is dumped from cars on the track on top of the bins. The ore is drawn off through gates in the bottom, operated by rack and pinion and is carried away on a conveyor. The side plates are ¼" thick and have 7" [stiffeners spaced about 4 feet apart. The hopper plates are ⅜" thick and are stiffened with 10" [s. A cut of this bin is given in Chapter VIII. The details are clearly shown in Fig. 55 and Fig. 103.

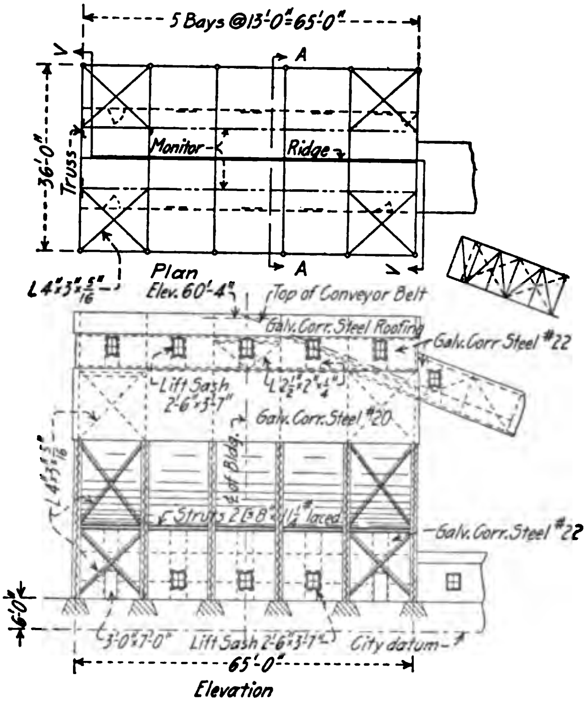


FIG. 102. COAL BUNKERS, RAPID TRANSIT SUBWAY.

Details of Hopper Coal Bins.—Detail drawings of a hopper coal bin designed by the Link Belt Engineering Co., Nicetown, Pa., are shown in Figs. 104 to 106. The plan of the hopper and detail drawings of the side plates are shown in Fig. 104; the detail drawings of the end plates are given in Fig. 105; and the details of supporting columns, the spouts, and miscellaneous details are shown in Fig. 106. This bin is designed to supply two furnaces having automatic stokers. The details are well worked out and are excellent examples of bin design. It will be seen that each plate, angle, etc., is completely detailed and is given a distinct erection mark. The stiffeners are placed on the outside of the bin only, and have fillers to avoid crimping the

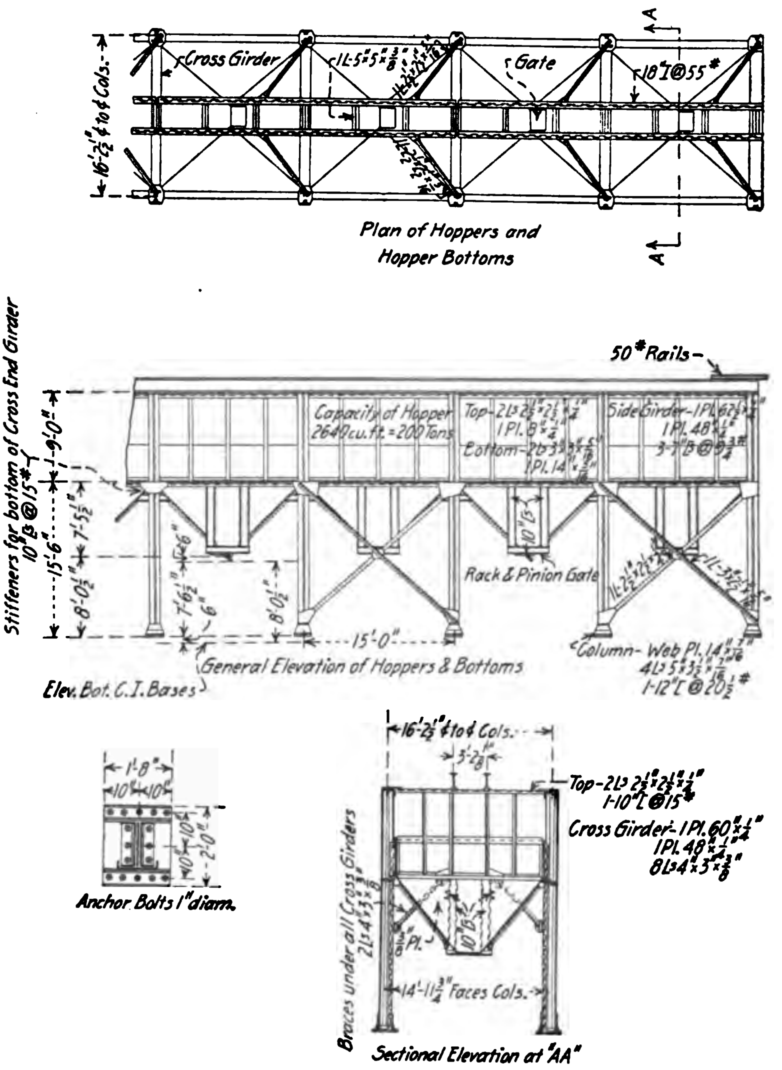


FIG. 103. HOPPER BIN CANANEA CONSOLIDATED COPPER CO., CANANEA, MEXICO.

angles. The side and hopper plates are ¼" thick. The bin is braced as shown in Fig. 104. The gate for this bin is shown in Fig. 97 a.

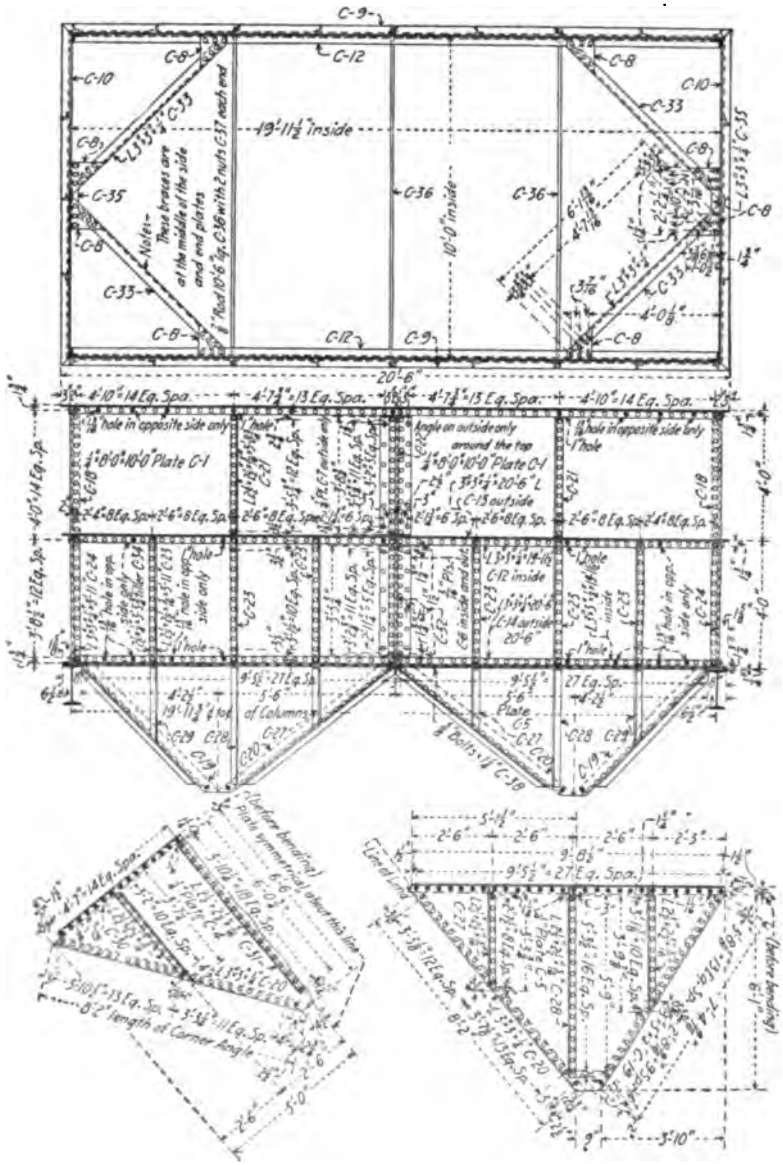


Fig. 104. Details of Hopper Bin.

Coal Storage Plant, Cincinnati Water Works.—The coal storage house, Fig. 107, consists of a steel bin 225 × 69 feet × 17 feet deep, supported on columns which rest on concrete pedestals. Between the columns are hopper bottoms which are provided with valves for discharging the coal into cars which are pushed by hand into the boiler house. The bins are filled from cars which are pushed in on the suspended floor. The outside columns are braced to take the thrust due to the coal. The sides of the bin are lined with $\frac{5}{16}''$ plates, while the hopper bottoms are made of $\frac{3}{8}''$ plates. The details are shown in Fig. 107.

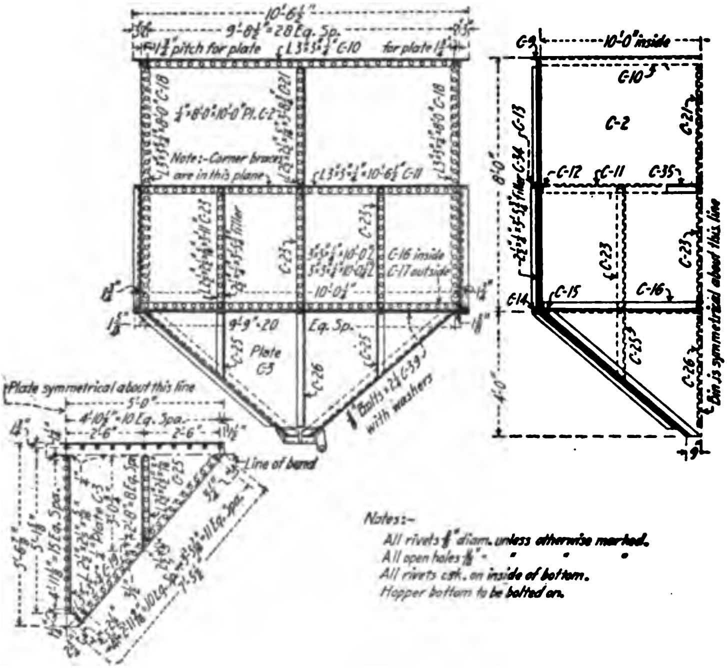

10'-0" inside
Note:- Corner bracing are in this plane
Plate symmetrical about this line
Line of bend
Bolts with washers
Bin is symmetrical about this line
C-2
C-10
C-11
C-12
C-15
C-16
C-25
C-26
Plate C-3
Eq. Sp.
Notes:-
All rivets ⅝" diam. unless otherwise marked.
All open holes 11/16" " " "
All rivets csk. on inside of bottom.
Hopper bottom to be bolted on.

FIG. 105. DETAILS OF HOPPER BIN.

For design of a steel hopper bin for coke and stone see Fig. 56.

Reinforced Concrete Coal and Coke Bins for Key City Gas Co.—The coal and coke bins built for the Key City Gas Co., Dubuque, Iowa, by the Expanded Metal and Corrugated Bar Co., St. Louis, Mo., are excellent examples of reinforced concrete bin construction. The structure is 33′ 8″ wide, 148′ 0″ long, and 38′ 3″ high.

A half plan of the bins showing the arrangement and the size of the bins is given in Fig. 109. A cross section and a longitudinal section

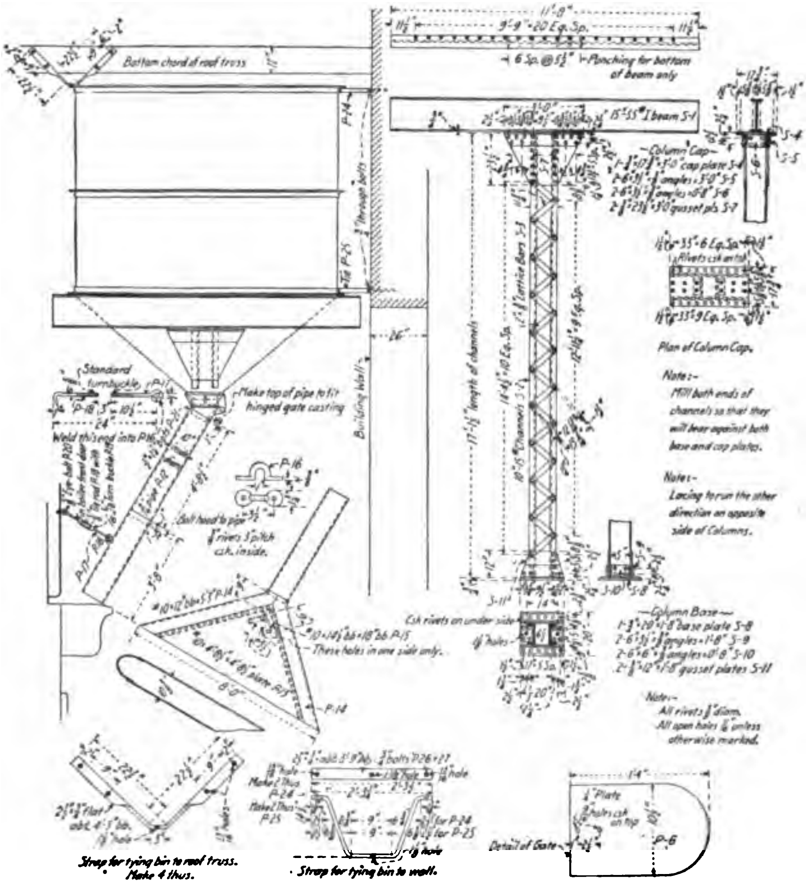
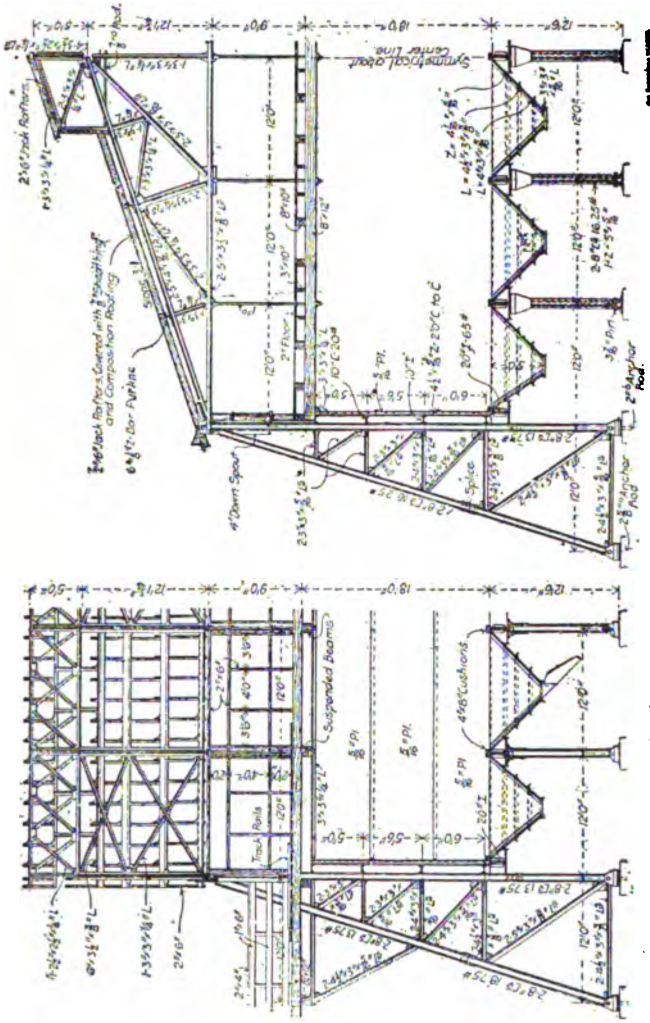

Bottom chord of roof truss
Punching for bottom of beam only
15″-55# I beam S-1
—Column Cap—
1-½″×17½″×3′-0″ cap plate S-4
2-6″×3½″×½″ angles×3′-0″ S-5
2-6″×3½″×½″ angles×0′-8″ S-6
2-½″×23½″×3′-0″ gusset pls. S-7
Rivets csk on top
Plan of Column Cap.
Note:-
Mill both ends of channels so that they will bear against both base and cap plates.
Note:-
Lacing to run the other direction on opposite side of Columns.
—Column Base—
1-¾″×20″×1′-8″ base plate S-8
2-6″×3½″×½″ angles×1′-8″ S-9
2-6″×6″×½″ angles×0′-8″ S-10
2-½″×12″×1′-8″ gusset plates S-11
Note:-
All rivets ¾″ diam.
All open holes 13/16″ unless otherwise marked.
Building Wall
Standard turnbuckle
24″
Weld this end into P-16
Make top of pipe to fit hinged gate casting
Bolt head to pipe
⅝″ rivets 3″ pitch csk. inside.
These holes in one side only.
P-14
Csk rivets on under side
Make 2 thus
P-24
P-25
Detail of Gate
¼″ Plate
P-6
Strap for tying bin to roof truss.
Make 4 thus.
Strap for tying bin to wall.

FIG. 106. DETAILS OF HOPPER BIN.

of the bins showing the details of the bins and the thickness of the concrete and the reinforcement are given in Fig. 108. The concrete was 1–2–5 Portland cement concrete, for which the modulus of elasticity was taken equal to 2,800,000, and the ultimate compressive stress equal to 2,700 lbs. per sq. in. The ultimate tensile strength of the steel was assumed as 55,000 lbs. per sq. in. The beams were designed

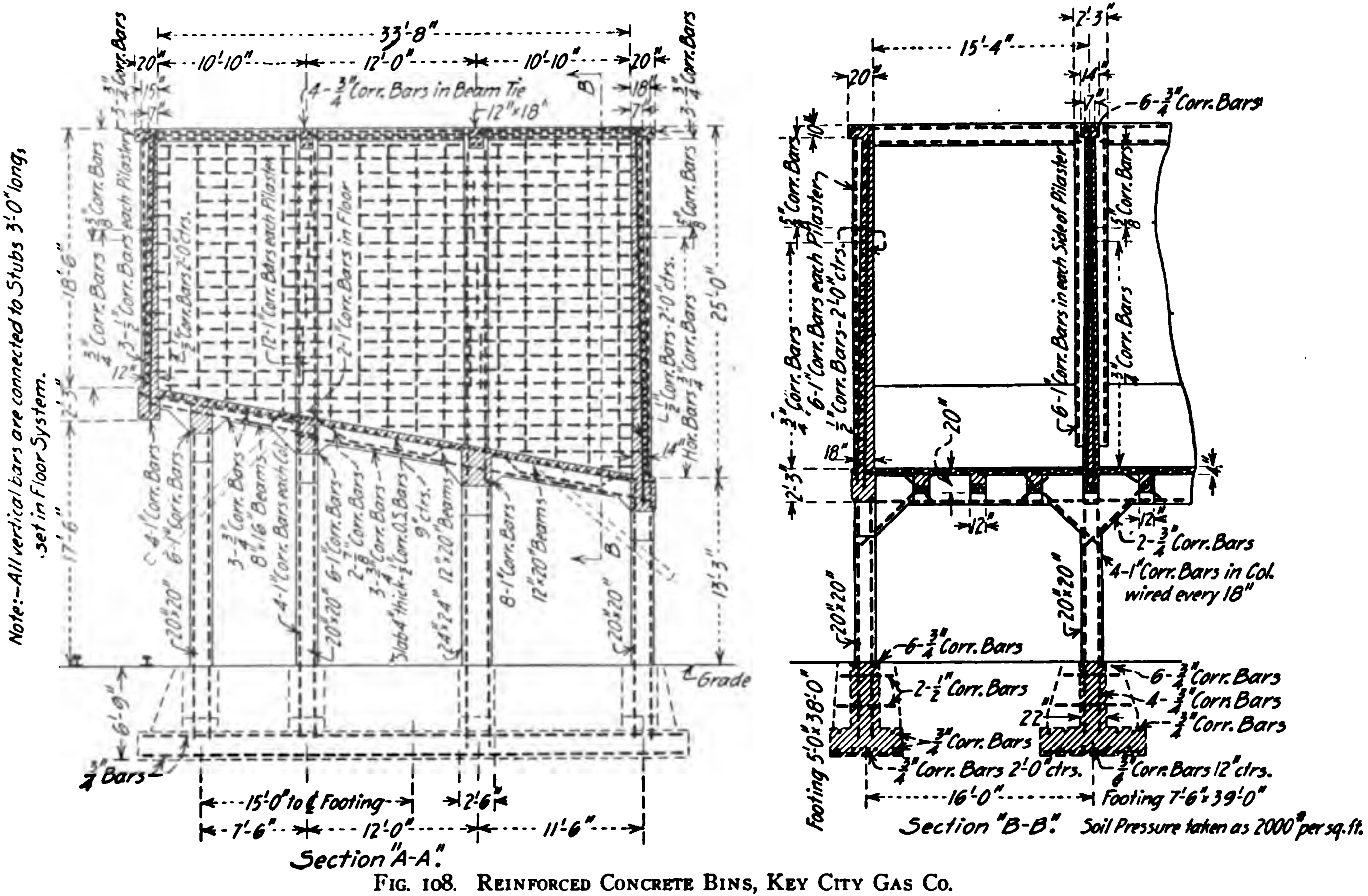


Typical Half Cross-Section

Part Longitudinal Section

FIG. 107. COAL STORAGE FOR CINCINNATI WATER WORKS.

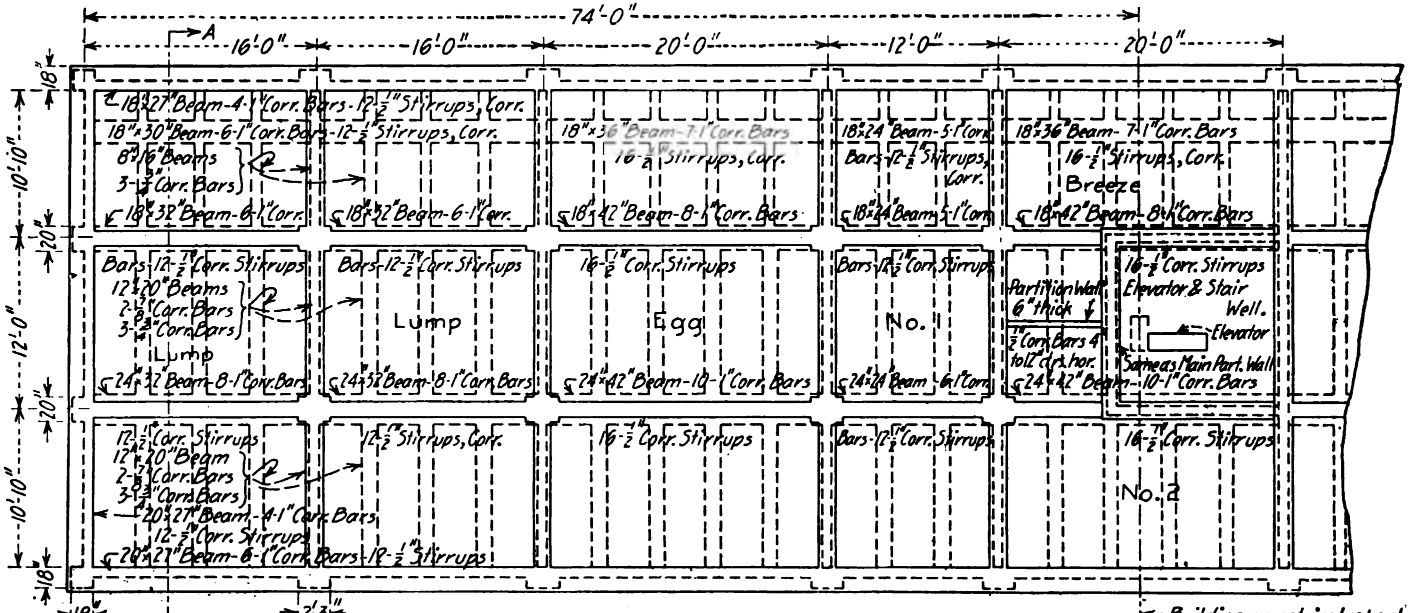
for a factor of safety of 4 in both concrete and steel, A. L. Johnson's formula being used. Coal was assumed to weigh 50 lbs. per sq. ft., and coke to weigh 32 lbs. per sq. ft.



Section "A-A."

Section "B-B."

FIG. 108. REINFORCED CONCRETE BINS, KEY CITY GAS CO.


74'-0"
16'-0"
16'-0"
20'-0"
12'-0"
20'-0"
A
A
18"
10'-10"
20"
12'-0"
20"
10'-10"
18"
18"
2'-3"
18"x27" Beam-4-1" Corr. Bars-12-½" Stirrups, Corr.
18"x30" Beam-6-1" Corr. Bars-12-½" Stirrups, Corr.
8"x16" Beams 3-¾" Corr. Bars
18"x32" Beam-6-1" Corr.
18"x32" Beam-6-1" Corr.
18"x36" Beam-7-1" Corr. Bars 16-½" Stirrups, Corr.
18"x42" Beam-8-1" Corr. Bars
18"x24" Beam-5-1" Corr. Bars-12-½" Stirrups, Corr.
18"x24" Beam-5-1" Corr.
18"x36" Beam-7-1" Corr. Bars 16-½" Stirrups, Corr.
Breeze
18"x42" Beam-8-1" Corr. Bars
Bars-12-½" Corr. Stirrups
12"x20" Beams 2-⅞" Corr. Bars 3-¾" Corr. Bars
Lump
24"x32" Beam-8-1" Corr. Bars
Bars-12-½" Corr. Stirrups
Lump
24"x32" Beam-8-1" Corr. Bars
16-½" Corr. Stirrups
Egg
24"x42" Beam-10-1" Corr. Bars
Bars-12-½" Corr. Stirrups
No. 1
24"x24" Beam-6-1" Corr.
Partition Wall 6" thick
½" Corr. Bars 4 to 12" c.-c. hor.
24"x42" Beam-10-1" Corr. Bars
16-½" Corr. Stirrups
Elevator & Stair Well.
Elevator
Same as Main Part. Wall
12-½" Corr. Stirrups
12"x20" Beam 2-⅞" Corr. Bars 3-¾" Corr. Bars
20"x27" Beam-4-1" Corr. Bars
12-½" Corr. Stirrups
20"x27" Beam-6-1" Corr. Bars-12-½" Stirrups
12-½" Stirrups, Corr.
16-½" Corr. Stirrups
Bars-12-½" Corr. Stirrups
16-½" Corr. Stirrups
No. 2

Plan of Top, showing Floor Beams and Partitions.

Building symetrical about this line except Elevator Well as shown.

FIG. 109. PLAN REINFORCED CONCRETE BINS, KEY CITY GAS CO.

Steel and Concrete Coal Storage Plant.*—The coal storage plant of the Lowell Gas Light Company, Lowell, Mass., is 500 feet long and 65 feet wide and has a storage capacity of 25,000 tons. The steel transverse bents are spaced 25′ 0″ centers. A cross section is shown in Fig. 110. The plant is a steel frame with concrete filled walls, concrete foundations and floor, and a tile roof supported on steel purlins. The side columns are braced frameworks and carry the pressure of the coal and the wind pressure. The roof is covered with non-glazed Ludowici tile laid on angle sub-purlins. Light is obtained through skylights in the roof made by substituting glass tile for the ordinary tile; ventilation is obtained through the opening between the roof and the wall. The steel work that would be exposed to coal is completely imbedded in concrete.

The conveyor is of the gravity bucket type with buckets 18 inches wide by 24 inches long, pivoted in the chain in such a manner that while they maintain themselves normally in an upright position, they can turn through a complete revolution. The chain is composed of double flat links of steel, and the conveyor moves about 40 feet per

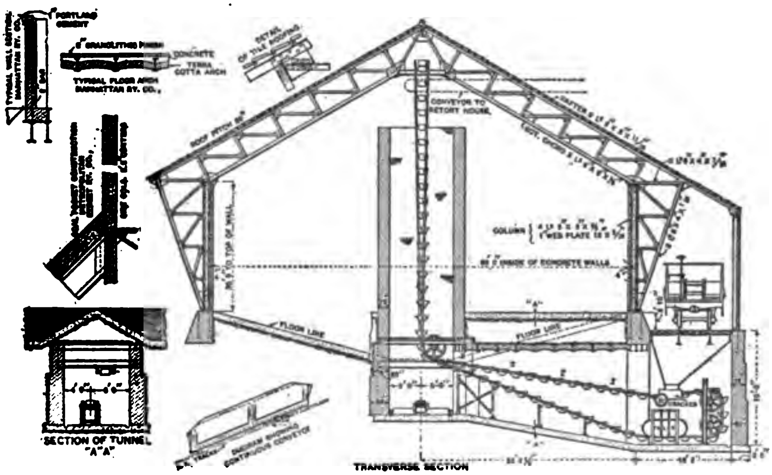


Fig. 110. Steel and Concrete Coal Storage Plant, Lowell, Mass.

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* *Trans. Am. Soc. M. E.*, May 1902.

minute. The machinery receives the coal from the railroad cars, cracks the lumps to a uniform size and carries them to any part of the building without transfer.

In removing the coal from storage it is drawn into the conveyor through a filler which is movable and which is placed under valves placed between columns 25′ 0″ apart. The openings are through buckle plates 10 feet square which may be removed to facilitate rapid emptying in case of fire from spontaneous combustion. Wrought iron pipes run through the coal to facilitate the determination of the temperature of the coal.

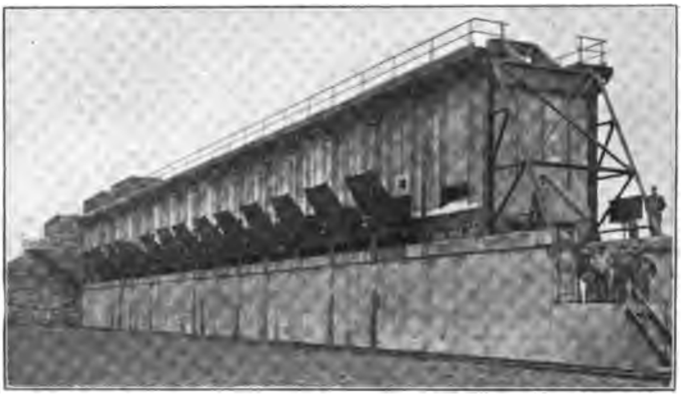
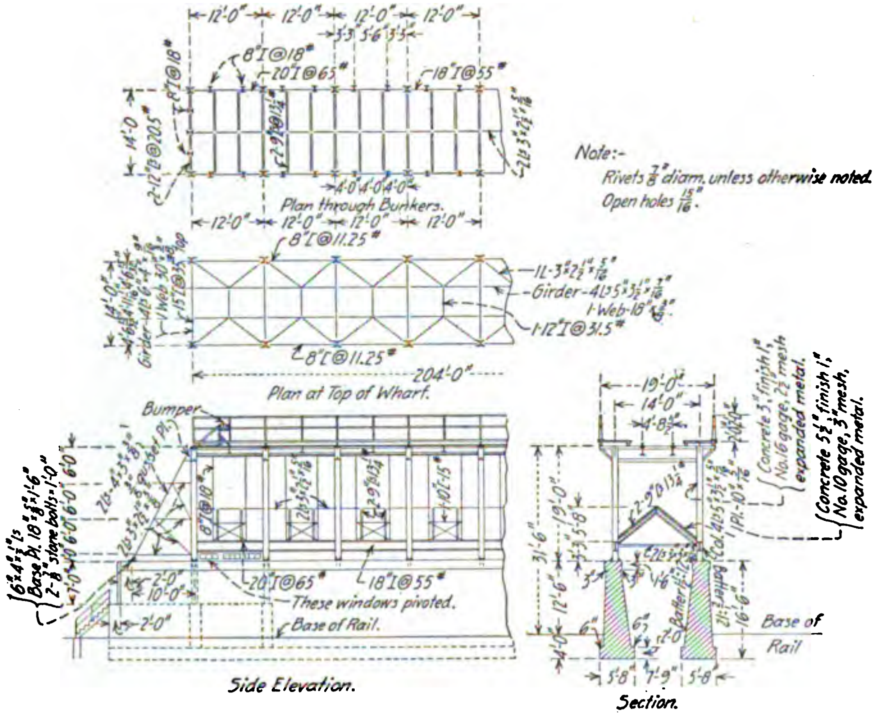


Fig. 111. Coaling Station, Morrisville, Pa.

Coaling Station at Morrisville, Pa.—The coaling station of the P. R. R. Co. at Morrisville, Pa., is made of a steel framework with reinforced concrete sides and lining. The coaling station is 204′ 0″ long, 14′ 0″ wide and 19′ 0″ above the top of the concrete foundations, the top of the foundations being 12′ 6″ above the base of rail and 16′ 6″ above the footing. The bins are 14′ 0″ wide and 12′ 0″ long, with a double hoppered bottom. The details of the steel framework are shown in Fig. 112. The finished coaling station is shown in Fig. 111. The specifications for the reinforced concrete were as follows:

Reinforced Concrete.—Reinforced concrete to include the sides and bottom of bins. All exposed sides and bottoms of bins, including columns shall be inclosed in concrete covered at least 1½″, the concrete to be composed of Portland cement 1 part, clean, sharp sand 2½ parts, and broken stone not larger than ¾″ in diameter 5 parts. The concrete to be well mixed and thoroughly rammed. The slabs for the bottom to be 5½″ thick, not including the finish (which shall be 1″ thick), and



Note:-
Rivets ⅞″ diam. unless otherwise noted.
Open holes 15/16″.

Fig. 112. Details of Steel Framework for Coaling Station, Morrisville, Pa.

to be reinforced with No. 10 gage 3″ mesh expanded metal; the slabs for the sides to be 3″ thick, not including the finish (which shall be 1″ thick), and to be reinforced with No. 16 gage 2½″ expanded metal. The inside of the bunkers to be finished with a granolithic top coat 1″ thick composed of Portland cement 1 part, clean, sharp sand 1 part, and fine crushed granite 1 part. The outside surface to be finished by making a perfectly smooth surface by finishing the same with a thin coat of cement.

Gates.—The gates were manufactured by the Link Belt Engineering Co. The Link Belt coaling chute is a patented device consisting of an under-cut gate and a swinging apron. The gate sides project upward in the form of a toothed segment and mesh with spur pinions mounted on a through shaft to which is keyed a combination hand and chain wheel. This combination facilitates operation and prevents the straining and bending to which gates of large capacity are subject. The gate is opened by a rotary downward swing and is closed instantly by an upward, or cutting off movement that is not retarded by the flowing coal, the chute being open at the top prevents danger from jamming. The flow being cut off permits the apron to be raised out of the way

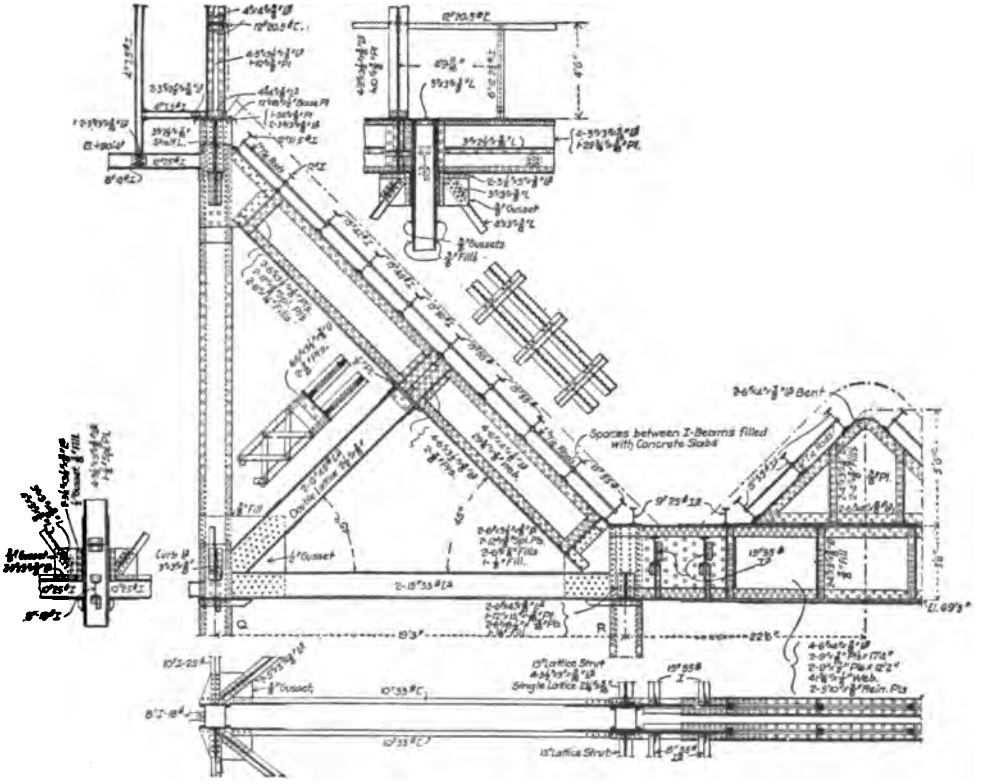


FIG. 113. TRANSVERSE BENT COAL BIN, INTERBOROUGH RAPID TRANSIT CO.

of passing locomotives and trains. It is held in position by counter weights. A hand chain permits control from the ground.

For a timber coaling station, see Fig. 132.

Coal Bin, Power House, Interborough Rapid Transit Co.—The coal bins are 61′ 0″ wide and extend nearly the full length of the power station. The bins have a W cross section as shown in Fig. 113. The transverse trusses are spaced 4′ 11″ and 15′ 6½″, and are supported on four rows of columns. The transverse trusses carry 15-in. I-beams spaced 3′ 0″ centers. These beams are connected by tie rods and have concrete arches with expanded metal reinforcement. The concrete arches are filled to a depth of 1½″ above the tops of the flanges and a finish coat of one to one Portland cement mortar 2″ thick is put on top of the arch filling. The concrete sides are carried up 4′ 6″ above the sloping sides. The bins are designed for a load of 45 tons per lineal foot of bin. The dead load was taken at 150 lbs. per sq. ft. for the sloping sides and 200 lbs. per sq. ft. for the bottom. A transverse section is shown in Fig. 113, and a view of the end framing of

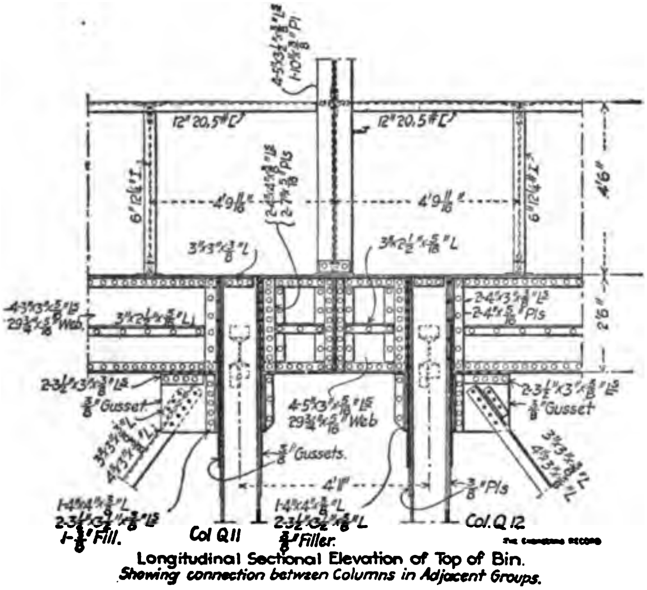


FIG. 114. LONGITUDINAL SECTION COAL BIN, INTERBOROUGH RAPID TRANSIT CO.

a bin is shown in Fig. 115. The girders and beams are made massive and amply strong.

The stresses in this bin were calculated as for the coal bins of the Ninety-sixth St. Station (see *Engineering Record*, September 1, 1900).

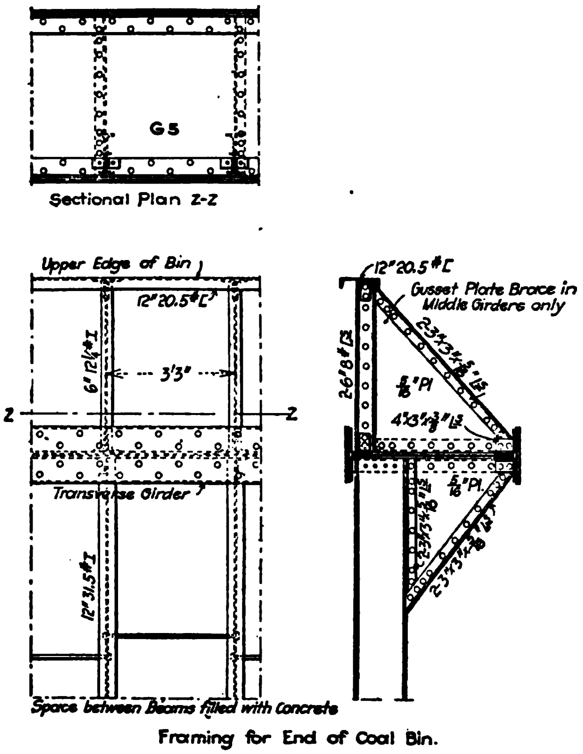
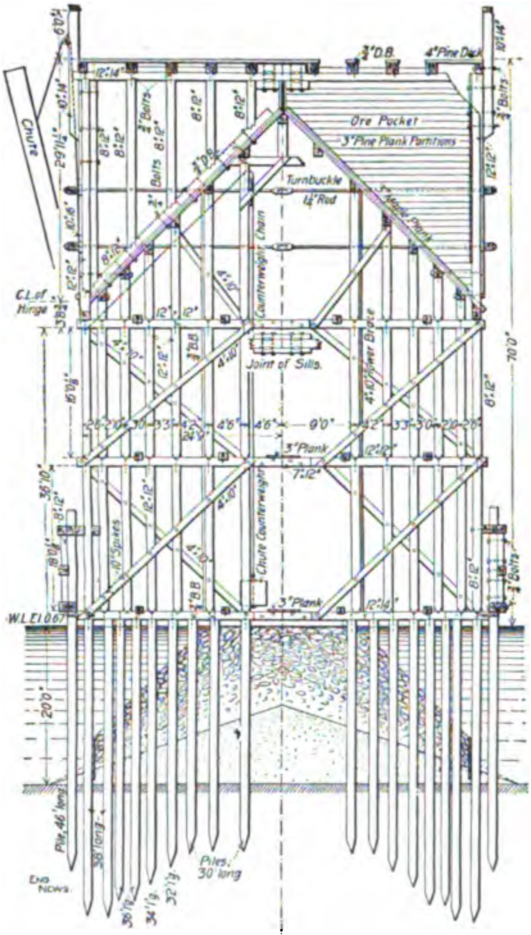
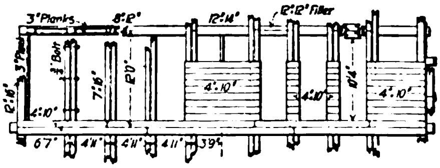


FIG. 115. END FRAMING COAL BIN, INTERBOROUGH RAPID TRANSIT CO.

Ore Docks, Chicago & Northwestern Ry., Escanaba, Mich.—The ore dock is a timber structure 1,920 feet long, 50 feet 2 inches wide, 70 feet high from the water to the dock. It has 320 pockets with a storage capacity of 58,000 tons. The construction is shown very clearly in Fig. 116 and Fig. 117. Rows of piles 30 to 46 feet long were driven in 20 feet of water, the bents being 6 feet apart, with 18 and 20 piles in alternate bents. Just above the water surface these

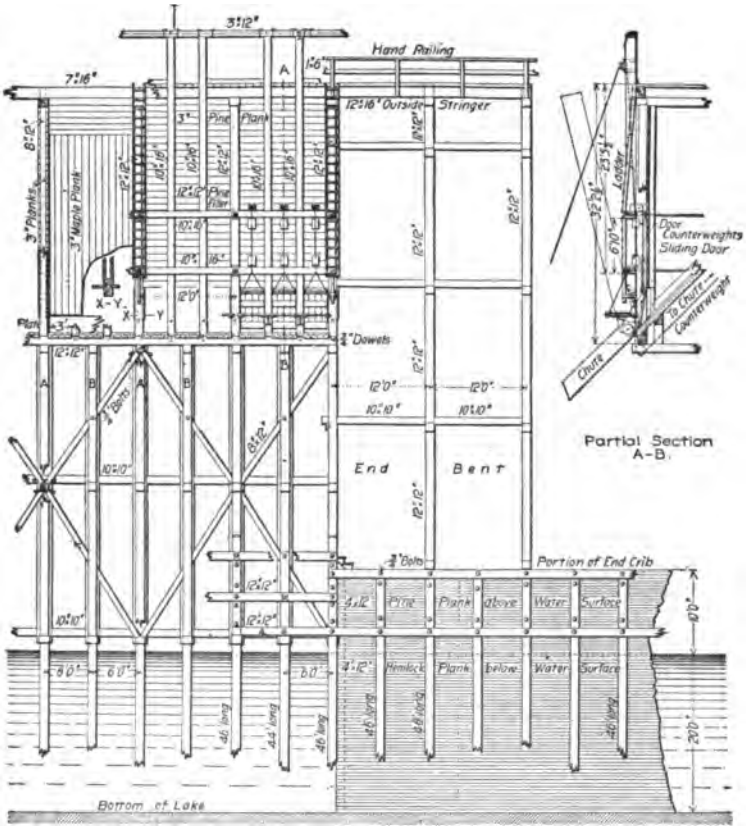


Half Section at Bents "A". Half Section at Bents "B".

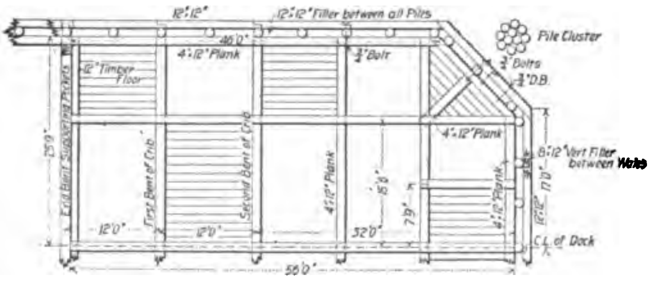


Deck Plan

FIG. 116. ORE DOCKS, ECANABA, MICH.



Longitudinal Section



Half Plan of End Crib

FIG. 117. ORE DOCKS, ECANABA, MICH.

piles are capped with timbers 12 × 14 in., forming the sills for the framed bents of the dock. The posts and caps are largely of timber 12 × 12 in., and 8 × 12 in., with longitudinal diagonals 8 × 12 in., and transverse diagonals or sway braces 4 × 10 in. These timbers are bolted and drift bolted together. Two rows of 1¼-in. transverse bars, set up by turn-buckles, pass through the pockets and resist the bursting pressure of the ore upon the face of the dock.

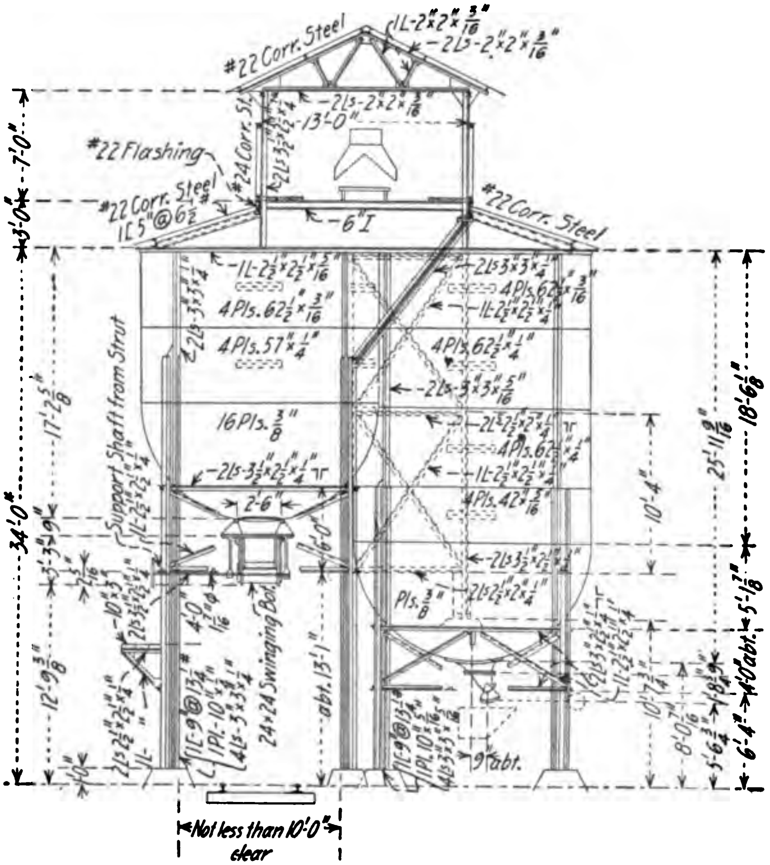


FIG. 118. ELEVATION CIRCULAR STEEL ORE BIN FOR OLD DOMINION COPPER MINING CO.

The pockets have sloping bottoms at an angle of 45° sheathed with 3-in. maple plank laid lengthwise up and down the slope. The partitions in front of the pockets are of 3-in. pine plank. Vertical sliding doors cover the openings, through which the ore flows to the chutes. These doors are counterweighted. The chutes are also balanced, the chain from the end of the chute passing up to a pulley above the dock,

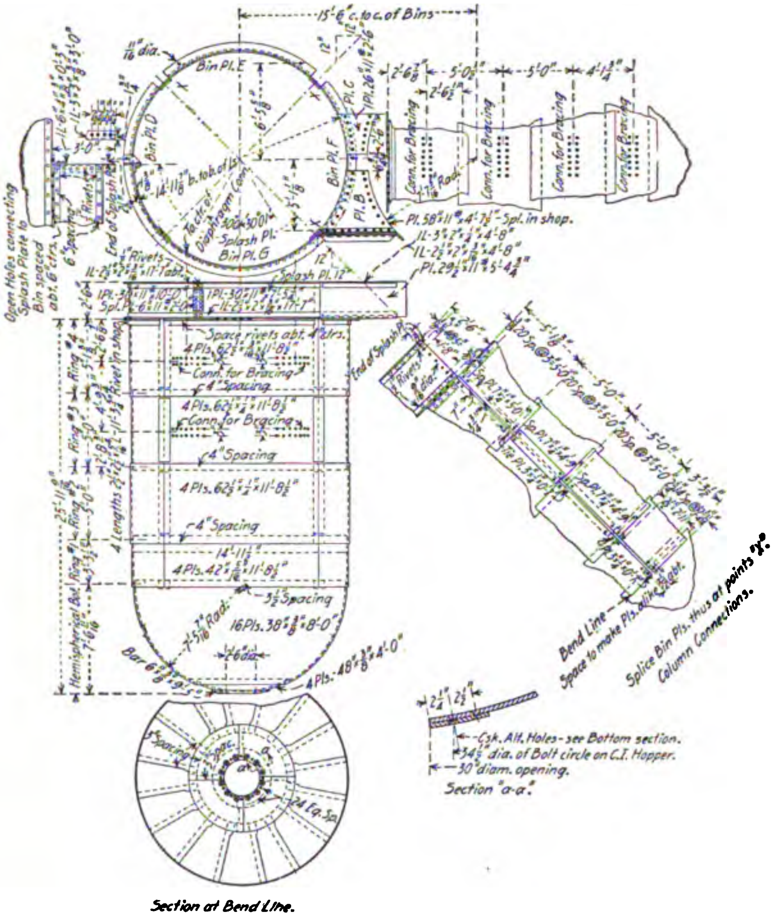
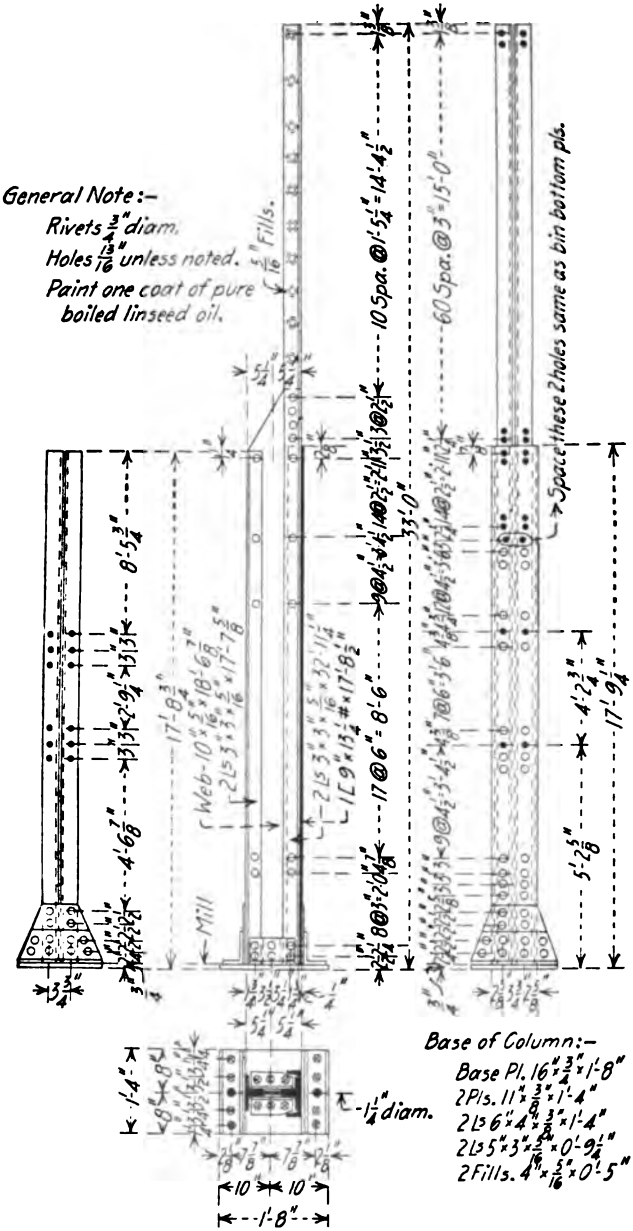

15'-6" c. to c. of Bins
11/16" dia.
Bin Pl. E
Bin Pl. D
Bin Pl. F
Pl. C
Pl. B
Bin Pl. G
Splash Pl.
Splash Pl. 12"
Tack ctr. of Diaphragm Conn.
14'-11 7/8" b. to b. of Ls.
End of Splash Pl.
Rivets
6" Spacing
Open Holes connecting Splash Plate to Bin spaced abt. 6" ctrs.
Conn. for Bracing
Pl. 58"×11/16"×4'-7 1/2" - Spl. in shop.
Pl. 29 1/2"×11/16"×5'-4 3/4"
Space rivets abt. 4" ctrs.
4 Pls. 62 1/2"×1/4"×11'-8 1/2"
4" Spacing
Ring #4
Ring #3
Ring #2
Ring #1
Rivet in shop
4 Lengths
14'-11 1/8"
4 Pls. 42"×5/16"×11'-8 1/2"
3 1/2" Spacing
16 Pls. 38"×3/8"×8'-0"
Rad.
Bar
2'-6" dia.
4 Pls. - 48"×3/8"×4'-0"
Hemispherical Bot.
25'-11"
7'-6 1/16"
Bend Line
Space to make Pls. alike abt.
Splice Bin Pls. thus at points "x." Column Connections.
Csk. All. Holes - see Bottom section.
34 1/2" dia. of Bolt circle on C.I. Hopper.
30" diam. opening.
Section "a-a."
Section at Bend Line.

FIG. 119. DETAILS FOR CIRCULAR BINS FOR OLD DOMINION COPPER MINING CO.

down to a pulley below the pocket, up under the pocket to another pulley, then down to a circular weight which hangs near the water line.

3. CIRCULAR BINS. Ore Bin for Old Dominion Copper Mining and Smelting Co.—The circular ore bin shown in Fig. 118, Fig. 119,



General Note:-
Rivets $\frac{3}{4}$" diam.
Holes $\frac{13}{16}$" unless noted.
Paint one coat of pure boiled linseed oil.

Base of Column:-
Base Pl. 16"×$\frac{3}{4}$"×1'-8"
2 Pls. 11"×$\frac{3}{8}$"×1'-4"
2 L's 6"×4"×$\frac{3}{8}$"×1'-4"
2 L's 5"×3"×$\frac{5}{16}$"×0'-9$\frac{1}{4}$"
2 Fills. 4"×$\frac{5}{16}$"×0'-5"

Fig. 120. Details of Columns for Circular Ore Bins for Old Dominion Copper Mining Co.

and Fig. 120 is one of 8 circular bins with hemispherical bottoms built by the Minneapolis Steel & Machinery Co. for the above company. In Fig. 118 it will be seen that the circular bins are of two heights, there being two large and six small ones. The ore is brought into the bin shed on side dump cars, and is drawn out through the bottom by means of gates as shown in Fig. 118. This type of gate is a very satisfactory one. Detail plans of one of the larger bins are given in Fig. 119, and are self-explanatory. Details of the columns supporting the bins are given in Fig. 120.

Cement Storage Bins, Illinois Steel Company, Chicago, Ill.—The storage plant consists of four cylindrical reinforced concrete bins, constructed on the Monier System. Details are given in Fig. 121, Fig. 122, and Fig. 123. The total capacity including the space between the bins is 25,000 barrels. The bins are 25 feet in diameter and are spaced 29 feet center to center, the space between each pair of adjacent walls being closed by a cylindrical shaft 30 inches in diameter, the entire structure being monolithic. The foundation is a continuous floor or bed of concrete 3 feet thick, 66 × 66 feet, with corners of 18 ft. 6 in. radius. About 5 inches above the base of the foundation is imbedded a netting of 9-in. mesh formed by $\frac{5}{8}$-in. round steel rods, tied together at their intersections by No. 18 wire. Upon this concrete bed is a series of piers 12 ft. 6 in. high, and 1 ft. 10 in. thick. Those near the outer surface of the tank are mainly 3 ft. 5 in. long, but others near the central portion are from 6 ft. 8 in. to 7 ft. 10 in. long. All the smaller piers have four steel rails imbedded in the concrete, the rails being connected by spacing bars riveted to them, and resting on $\frac{1}{2}$-in. steel plates imbedded in the concrete floor about 15 inches below the surface. The piers are capped with similar plates. The larger piers have six or eight rails. Upon the piers rest concrete-steel girders 15 inches deep and 4 feet wide, with vertical openings at intervals for the discharge spouts. Through each girder run four lines of horizontal steel bars near the top, and four other lines bent to form truss rods, with sheets of wire netting on each side of each set of bars.

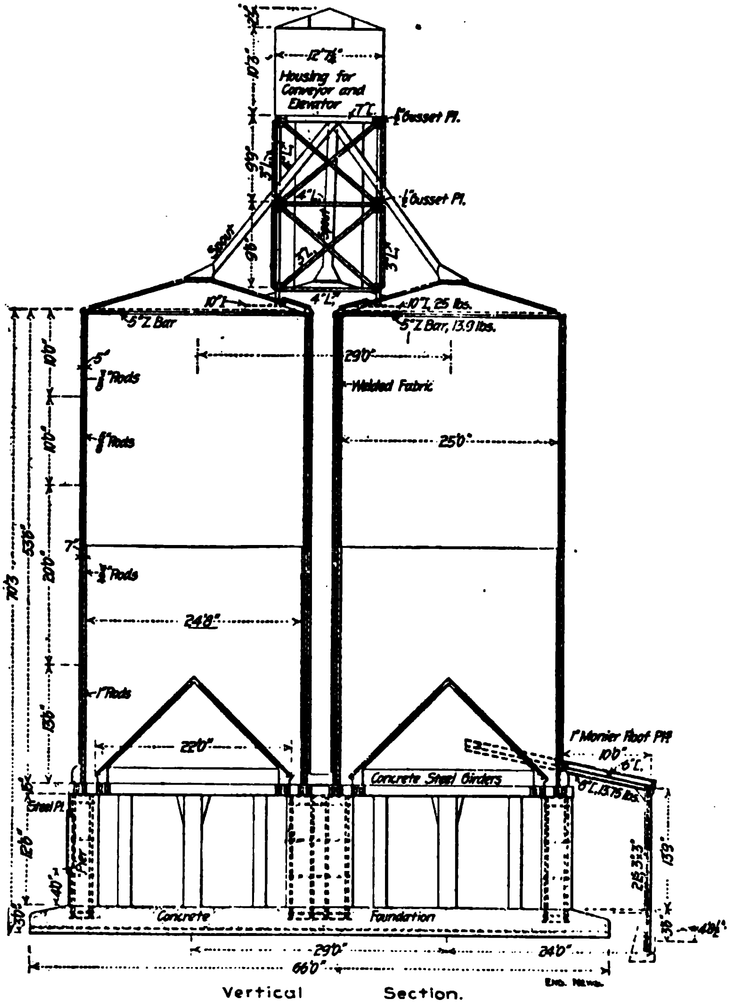


FIG. 121. CEMENT STORAGE BINS OF REINFORCED CONCRETE, MONIER CONSTRUCTION, ILLINOIS STEEL COMPANY, CHICAGO, ILL.

The cylindrical tanks, 53 ft. 6 in. high, rest upon this system of girders, the base being 13 feet 9 in. above the level of the concrete floor. The walls are 7 inches thick in the lower part and 5 inches thick in the upper part, the thickness being increased where they unite with the circular walls which close the spaces between the tanks.

Within the concrete walls is imbedded a continuous sheet of netting of No. 9 wires electrically welded at their intersections and forming rectangular meshes 1 × 4 inches. Around this (and alternately inside and outside of it) are horizontal rings of rods 4 inches apart, tied to the netting by wire. These rods vary from 1 inch in diameter near the base to ⅜ inch in diameter near the top, while the top is finished with a ring formed by a 5-in. Z-bar. The roof is conical, 2 inches thick, with a manhole near the edge and an opening at the top for the

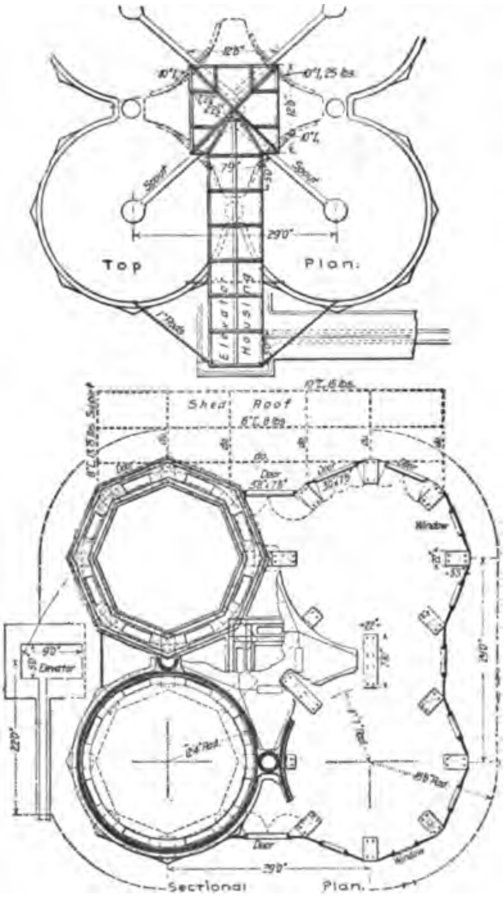


FIG. 122. PLAN OF REINFORCED CONCRETE CEMENT STORAGE BINS.

spout. It was thought that with a hopper bottom and central discharge, material such as cement would give trouble by bridging, and the bottom is, therefore, made conical with eight discharge openings in the circular space between the base of the cone and the side of the tank. These are about 15 × 18 inches, and each serves two spouts leading to conveyors which carry the cement to the packing and shipping department. The conical bottom is 4 inches thick, reinforced with bars and wire netting, and its diameter at the base is 22 feet.

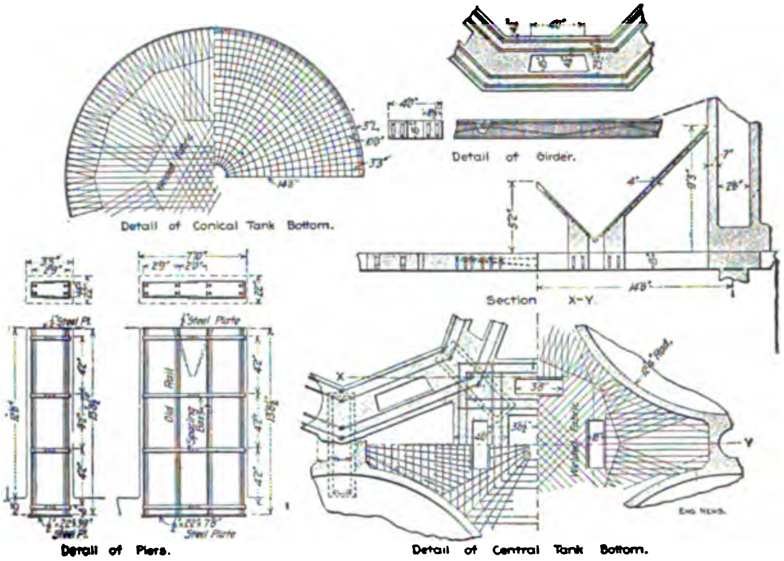


FIG. 123. DETAILS OF REINFORCED CONCRETE CEMENT STORAGE BINS.

The concrete in the foundation floor and piers is composed of 1 part Portland cement, 3 parts coarse sand, and 4 parts broken stone. That of the bins is composed of 1 part Portland cement, to 3½ parts sand, no broken stone being used. It was mixed moderately wet and lightly rammed in wooden forms. These forms were constructed of 3-inch plank 3 feet long, kept together by three horizontal angle irons supported by three vertical angle irons clamped together above the wall. The concrete was poured in and tamped inside the forms, which were raised in 45° sections 28 inches high, every 24 hours. The work was carried on by day and night so as to prevent any setting between the layers of concrete.

The cement is brought from the mill by a horizontal screw conveyor and delivered into the boot of a vertical bucket elevator, which again delivers it to a horizontal screw conveyor above the bins, by which it is carried to the spout of one or other of the bins. The elevator runs in a rectangular shaft 12 × 14 feet, and the distributing conveyor in a chamber or housing 10 ft. 3 in. × 12 ft. 4 in. supported on a steel bent. The shaft and housing are built of a framing of steel angles, sheathed with Monier concrete steel plates 2 × 5 feet × ⅝ in. thick, made of cement mortar and wire netting. The floor of the conveyor housing is of similar plates 1 inch thick.

Reinforced Concrete Sand Bins, New Brighton, N. J.—The plant consists of a reinforced concrete drying house 34 ft. × 52 ft. and 87 ft. high, and two sand bins 30 ft. inside diameter and 50 ft. high, with a capacity of 1140 tons of dry sand each.

The two bins are supported on fifteen reinforced concrete columns. The center column and the six columns of the center ring of each bin are 22 inches square, while the columns in the outer ring are 18 × 22 inches, and all are reinforced with four ¾-in. vertical bars.

The side walls are 10 inches thick and are reinforced with horizontal bars with ends overlapping 24 inches, and of sizes as shown in Fig. 124. The walls are also reinforced by two sets of vertical bars; the lower set of ½-in. bars are 11 feet long and are spaced 2 ft. 6 in. apart, while the upper set of ⅜-in. bars are 14 ft. 6 in. long spaced 2 ft. 6 in. apart. The bars overlap 6 inches. The walls were built in 5-foot courses, which were bonded together by an additional set of ⅜-in. bars, 2 ft. long and spaced 2 ft. 6 in. The bottom is 13 inches thick and is reinforced with circular and inclined radial bars as noted in the drawing. The molds for the circular bins were made with planed vertical staves 1⅛ inches thick, forming panels 5 feet high, and about 8 feet long. The staves were nailed to 2 × 10-inch horizontal segmental ribs, one at the upper edge, one at the lower edge, and one at the center of each panel. The ribs were arranged so as to lap with the ribs of the adjacent panels to which they were bolted. The ribs were braced by inside and outside vertical standards 4 feet apart on centers, each made

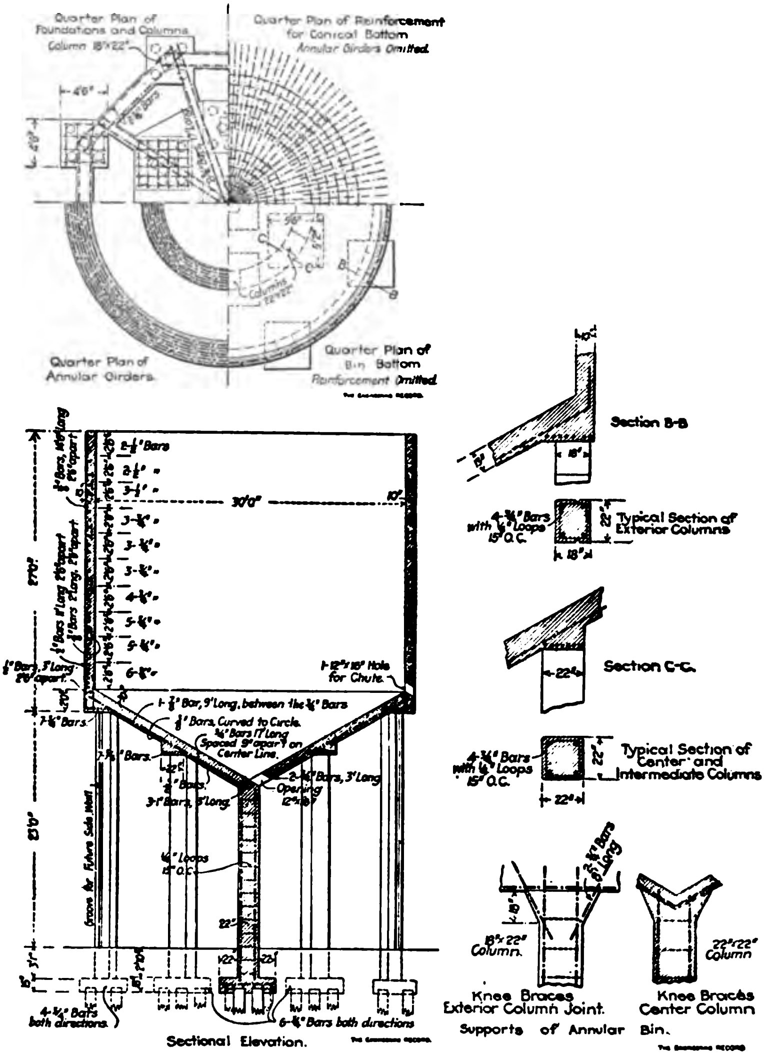

Quarter Plan of Foundations and Columns
Column 18"×22"
4'6"
4'6"
Quarter Plan of Reinforcement for Conical Bottom
Annular Girders Omitted
Columns 22"×22"
B
C
Quarter Plan of Annular Girders.
Quarter Plan of Bin Bottom
Reinforcement Omitted
The Engineering Record.
27'0"
13'0"
30'0"
10"
2-½" Bars
2-½" "
3-½" "
3-⅝" "
3-⅝" "
3-⅝" "
4-⅝" "
5-⅝" "
5-⅝" "
6-⅝" "
1-12"×18" Hole for Chute.
1-⅞" Bar, 9' Long, between the ¾" Bars
⅞" Bars, Curved to Circle.
¾" Bars 17' Long Spaced 9" apart on Center Line.
7-¾" Bars.
2-¾" Bars, 3' Long
Opening 12"×18"
3-1" Bars, 5' Long.
¾" Loops 15" O.C.
22"
Groove for Future Side Wall
4-¾" Bars both directions.
6-¾" Bars both directions
Sectional Elevation.
Section B-B
18"
Typical Section of Exterior Columns
4-¾" Bars with ¼" Loops 15" O.C.
22"
18"
Section C-C.
22"
Typical Section of Center and Intermediate Columns
4-¾" Bars with ¼" Loops 15" O.C.
22"
22"
2-¾" Bars 6' Long
18"×22" Column.
22"×22" Column
Knee Braces Exterior Column Joint.
Knee Braces Center Column
Supports of Annular Bin.
The Engineering Record.

FIG. 124. REINFORCED CONCRETE SAND BIN, J. B. KING & CO., NEW BRIGHTON, STATEN ISLAND.

with a pair of 2 × 6-inch strips 6 feet long, fillered 1 inch apart and tied together by temporary radial bolts through the molds. These bolts passed through sleeves of No. 16 gage black sheet iron bent cold, permanently imbedded in the concrete and finally filled with mortar. Enough molds were made for one complete course for one bin and were therefore used ten times in constructing the two bins.

After the concrete was 24 hours old, the tie bolts were removed from the molds and they were lifted by a small hoist until the lower edges engaged the top of the concrete for about 2 inches and the vertical standards engaged it for about 1 foot. The molds were supported on blocks bolted to the wall through the sleeves in which the mold bolts had passed. The interior of the molds was thoroughly covered with soft soap to prevent adhesion of the concrete. Each 5-foot course was made monolithic by continuous concreting in one day's work. The bin bottom was made without an inside mold.

In calculating the bottom and walls of the bins, fluid pressure for dry sand weighing 100 lbs. per cu. ft. was assumed, and a working stress of 30,000 lbs. per square inch allowed on steel bars.

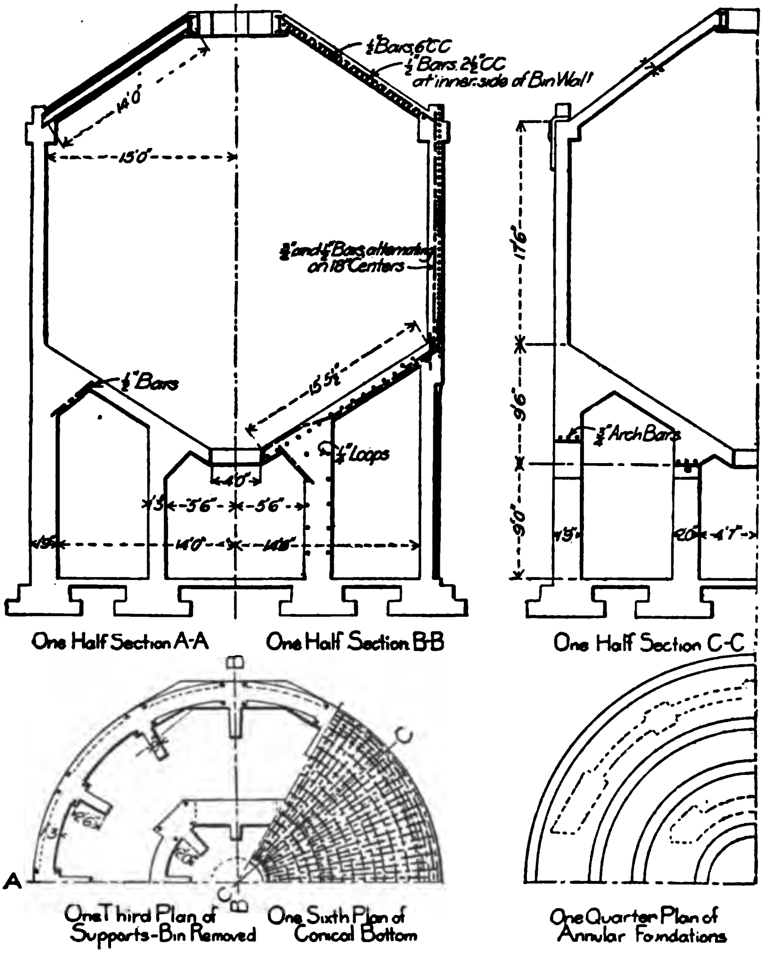
Reinforced Concrete Coal Pocket, Atlantic City Water Works.—The coal bin is 30 feet in diameter with conical bottom and pyramidal roof as shown in Fig. 125. The capacity of the bin is 400 tons.

The concrete was made of 1 part Alpha Portland cement, 2½ parts bank sand, and 5 parts gravel. The sand and gravel both had 38 per cent voids. The gravel was cleaned and screened so as to be ¼ to ¾ in. in size for the shell of the coal bin, and ¼ to 2 in. in size for the other work. The concrete weighed exactly 150 lbs. per cu. ft.

The steel reinforcement was mild steel and was required to pass standard tests. Plain bars were used.

The molds for all parts above the foundation were of matched lumber, surfaced one side only. The rough side was placed on the side in contact with the concrete on interior surfaces, but on all other surfaces the planed side was placed next to the concrete. Concrete ingredients were added by actual measurements. The cement and sand were mixed dry, water was added and the whole mixed. The gravel was wet and thoroughly mixed with the mortar, and water added to make a wet mixture.

All surfaces above the foundation except the interior of the bin and the roof were given a coat of neat cement grout mixed with lampblack, applied with a brush. The roof was finished on top with 1 inch of 1 to 2 cement plaster applied while the cement was green.


One Half Section A-A

One Half Section B-B

One Half Section C-C

One Third Plan of Supports-Bin Removed

One Sixth Plan of Conical Bottom

One Quarter Plan of Annular Foundations

FIG. 125. REINFORCED CONCRETE COAL BIN, ATLANTIC CITY WATER WORKS.

The following are the approximate quantities in the work: Excavation, 233 cu. yds.; concrete, 317 cu. yds.; steel for reinforcement, 13,700 lbs.; steel beams, plates, etc., 3,250 lbs.; 3-in. down-spouts, 210 lin. ft.

The contract price of the coal bin was $3,795; of the elevating machinery $2,650; which, together with $37.50 for extra work, makes a total cost of $6,482.50.

The bin was designed by Kenneth Allen, city engineer.

Reinforced Concrete Sand Bins, Peerless Brick Co.—The reinforced concrete sand bins shown in Fig. 126 have a capacity of 2,200 cubic yards, and were built in 1905 for the Peerless Brick Co., New York, N. Y. The bin walls are 8 inches in thickness and are reinforced with vertical and horizontal Ransome bars. The vertical bars are ⅜ inch square and are spaced 18 inches centers. The horizontal bars vary from ⅜ × 1½ inch bars, spaced 8 to 12 inches centers, to ⅜ inch square bars, spaced 18 inches centers, as shown in Fig. 126. The vertical rods are in lengths of about 14 feet, lap 18 inches at the joints and are wired together. The horizontal bars are set just inside the vertical bars and are 1½ inches clear of outer surface. Their ends lap from 12 to 16 inches, and are separated by a ½-inch washer and connected by a ½-inch bolt at the center of each joint. The ends of the bars are bent outward 90°, and project 1 inch to secure additional grip in the concrete.

The lower part of the walls and the floor are waterproofed with 4-ply tar and felt. The roof is covered with tar and gravel. Other details are clearly shown in Fig. 126.

The concrete was composed of 1 part Lehigh Portland cement, 3 parts sand, and 5 parts gravel, mixed very wet in a Ransome mixer.

The forms for the cylindrical walls were made in segments about 9 feet long. They were 7 feet in height, the entire ring holding 42 cubic yards, which was filled in one day. The joints between the different fills were bonded by vertical ⅜-in. bars 3 feet long and 2 feet apart on centers. The inner and outer molds were each stiffened with three horizontal circular ribs, one in the center, one at the top and one at the bottom. The inside and outside molds were tied together by horizontal bolts. Radial knee-brace timbers about 10 feet apart

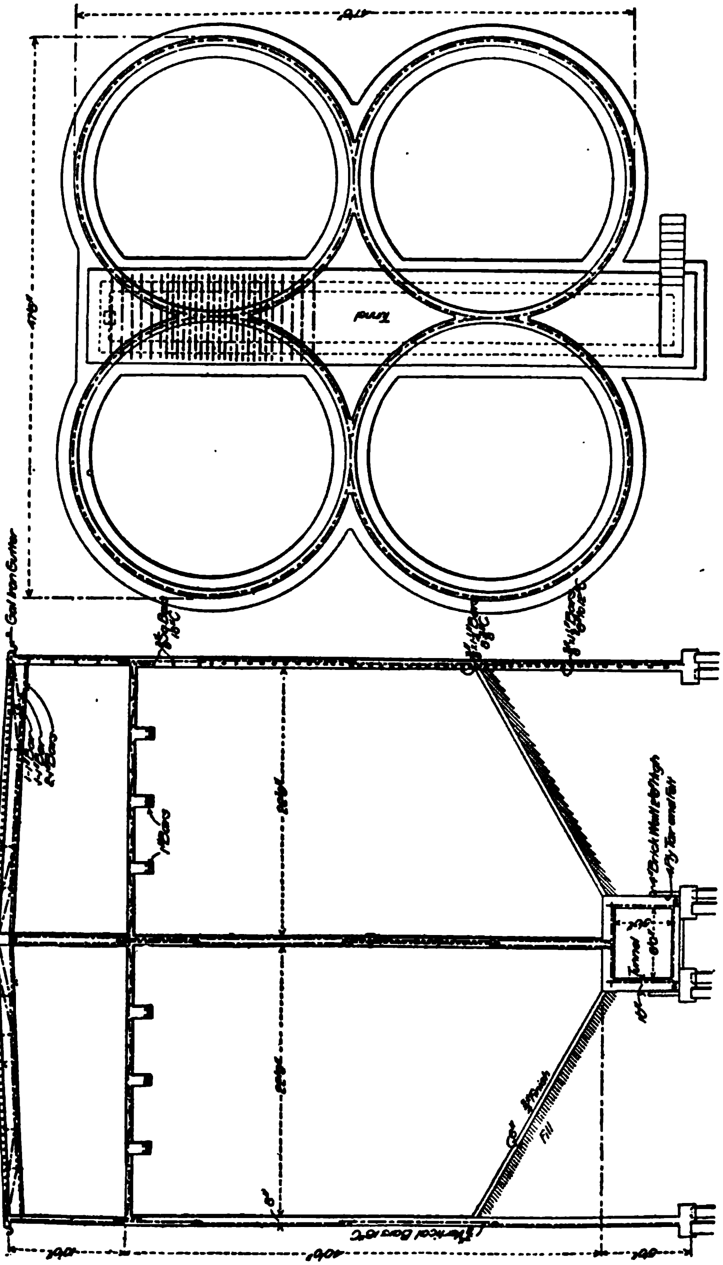


FIG. 126. REINFORCED CONCRETE SAND BINS FOR PEERLESS BRICK CO., NEW YORK, N. Y.

were bolted to the walls just below the top of the molds, and projected 4 feet beyond it, both inside and outside to carry the working platforms. After the completion of the wall these braces were removed and the holes left were carefully filled and pointed with mortar.

The bins were designed according to Janssen's formula (144).

The bins contain in all 680 cubic yards of concrete, and 4,510 lbs. of steel. They were built in about 60 working days with an average force of 11 carpenters and 14 laborers.

CHAPTER XIII.

COST OF BINS.

Introduction.—The cost of bins depends so much upon the design and upon local conditions that it is possible to give only average conditions that may be used as a guide to the experienced estimator. For methods of estimating the weight of steel in structures and for additional data on costs see the author's "The Design of Steel Mill Buildings."

COST OF STEEL BINS.—The cost of steel bins may be divided into (1) cost of material; (2) cost of fabrication; (3) cost of erection; and (4) cost of transportation.

1. **Cost of Material.**—The price of structural steel is quoted in cents per pound delivered free on board cars (f. o. b.) at the point at which quotation is made. Current prices may be obtained from the Engineering News, Iron Age, or other technical papers. Present prices (1906) f. o. b. Pittsburg, Pa., are as follows:

TABLE XXXI.

PRICES OF STRUCTURAL STEEL (1906), F. O. B. PITTSBURG, PA., IN CENTS PER POUND.

Material.	Price in Cents per Lb.
I beams 18" and over	1.70
I beams and channels 3" to 15"	1.60
Angles 3" to 6" inclusive	1.70
Angles over 6"	1.80
Tees 3" and over	1.80
Zees 3" and over	1.70
Channels, angles, Ts and Zs under 3"	1.60
Plates, structural, base	1.60
Plates, flange, base	1.70
Bars and rivet rods	1.60

Material.	Price in Cents per Lb.
Deck beams and bulb angles	1.90
Checkered plates	2.25
Forged rounds 5" to 11" diameter	2.75
Eye bar flats 8" to 12" inclusive	2.10
Eye bars over 6" and under 8"	1.90
Eye bars 6" and under	1.60

Eye bars over 12" wide subject to special arrangement.

Rolled rounds over 3" diameter, 18" long or over, 0.35 cents per lb. extra.

Rolled rounds over 3" diameter, under 18" long, 0.65 cents per lb. extra.

The prices above are net with the exception of those for plates and bars which are subject to standard extras as follows:

Extras.—*Shapes, Plates and Bars:*

(Cutting to length)

Under 3' to 2', inclusive	0.25	cts. per lb.
Under 2' to 1', inclusive	0.50	" "
Under 1'	1.55	" "

Extras—Plates (Card of January 7, 1902):

Base ¼" thick, 100" wide and under, rectangular (see sketches).

Weights—see Mfgr's. Standard Specifications, Carnegie or Cambria Hand-Books, or Appendix II.

	Per 100 Lbs.
Widths—100" to 110"	$ .05
110" to 115"	.10
115" to 120"	.15
120" to 125"	.25
125" to 130"	.50
Over 130"	1.00
Gages under ¼" to and including 3/16"	.10
Gages under 3/16" to and including No. 8	.15
Gages under No. 8 to and including No. 9	.25
Gages under No. 9 to and including No. 10	.30
Gages under No. 10 to and including No. 12	.40
Complete circles	.20
Boiler and flange steel	.10
Marine and fire box	.20
Ordinary sketches	.10

(Except straight taper plates, varying not more than 4″ in width at ends, narrowest end not less than 30″, which can be supplied at base prices.)

TABLE XXXII.

STANDARD CLASSIFICATION OF EXTRAS ON IRON AND STEEL BARS.*

Rounds and Squares.

Squares up to 4¼ inches only. Intermediate sizes take the next higher extra.

		Per 100 lbs.
$\frac{3}{4}$ to 3	inches	Rates.
$\frac{5}{8}$ to $\frac{11}{16}$	"	\$0.10 extra.
$\frac{1}{2}$ to $\frac{9}{16}$	"	.20 "
$\frac{7}{16}$	"	.40 "
$\frac{3}{8}$	"	.50 "
$\frac{5}{16}$	"	.60 "
$\frac{1}{4}$ and $\frac{9}{32}$	"	.70 "
$\frac{7}{32}$	"	1.00 "
$\frac{3}{16}$	"	2.00 "
$3\frac{1}{16}$ to $3\frac{1}{2}$	"	.15 "
$3\frac{9}{16}$ to 4	"	.25 "
$4\frac{1}{16}$ to $4\frac{1}{2}$	"	.30 "
$4\frac{9}{16}$ to 5	"	.40 "
$5\frac{1}{8}$ to $5\frac{1}{2}$	"	.50 "
$5\frac{5}{8}$ to 6	"	.75 "
$6\frac{1}{8}$ to $6\frac{1}{2}$	"	1.00 "
$6\frac{5}{8}$ to $7\frac{1}{4}$	"	1.25 "

Flat Bars and Heavy Bands.

					Per 100 lbs.
1 to 6	inches	x	$\frac{3}{8}$ to 1	inch	Rates.
1 to 6	"	x	$\frac{1}{4}$ and $\frac{5}{16}$	"	\$0.20 extra.
$\frac{11}{16}$ to $\frac{15}{16}$	"	x	$\frac{3}{8}$ to $\frac{3}{4}$	"	.40 "
$\frac{11}{16}$ to $\frac{15}{16}$	"	x	$\frac{1}{4}$ and $\frac{5}{16}$	"	.50 "
$\frac{9}{16}$ and $\frac{5}{8}$	"	x	$\frac{3}{8}$ to $\frac{1}{2}$	"	.50 "
$\frac{9}{16}$ and $\frac{5}{8}$	"	x	$\frac{1}{4}$ and $\frac{5}{16}$	"	.70 "
$\frac{1}{2}$	"	x	$\frac{3}{8}$ and $\frac{7}{16}$	"	.90 "
$\frac{1}{2}$	"	x	$\frac{1}{4}$ and $\frac{5}{16}$	"	1.10 "
$\frac{7}{16}$	"	x	$\frac{3}{8}$	"	1.00 "
$\frac{7}{16}$	"	x	$\frac{1}{4}$ and $\frac{5}{16}$	"	1.20 "
$\frac{3}{8}$	"	x	$\frac{1}{4}$ and $\frac{5}{16}$	"	1.50 "

---

* Adopted August, 1902.

	Per 100 lbs.
$1\frac{1}{8}$ to 6 inches x $1\frac{1}{16}$ to $1\frac{3}{16}$ inch	.10 “
$1\frac{1}{8}$ to 6 “ x $1\frac{1}{4}$ to $1\frac{1}{2}$ “	.20 “
$1\frac{3}{4}$ to 6 “ x $1\frac{5}{8}$ to $2\frac{3}{4}$ “	.30 “
$3\frac{1}{8}$ to 6 “ x 3 to 4 “	.40 “

Light Bars and Bands.

	Per 100 lbs.
$1\frac{1}{2}$ to 6 in. x Nos. 7, 8, 9 and $\frac{3}{16}$....................	$0.40 extra.
$1\frac{1}{2}$ to 6 in. x Nos. 10, 11, 12 and $\frac{1}{8}$ in................	.60 “
1 to $1\frac{7}{16}$ in. x Nos. 7, 8, 9 and $\frac{3}{16}$ in................	.50 “
1 to $1\frac{7}{16}$ in. x Nos. 10, 11, 12 and $\frac{1}{8}$ in................	.70 “
$\frac{13}{16}$ to $\frac{15}{16}$ in. x Nos. 7, 8, 9 and $\frac{3}{16}$ in................	.70 “
$\frac{13}{16}$ and $\frac{15}{16}$ in x Nos. 10, 11, 12 and $\frac{1}{8}$ in................	.80 “
$\frac{11}{16}$ and $\frac{3}{4}$ in. x Nos. 7, 8, 9 and $\frac{3}{16}$ in................	1.00 “
$\frac{11}{16}$ and $\frac{3}{4}$ in. x Nos. 10, 11, 12 and $\frac{1}{8}$ in..............	1.20 “
$\frac{9}{16}$ and $\frac{5}{8}$ in. x Nos. 7, 8, 9 and $\frac{3}{16}$ in..................	1.20 “
$\frac{9}{16}$ and $\frac{5}{8}$ in. x Nos. 10, 11, 12 and $\frac{1}{8}$ in..............	1.30 “
$\frac{1}{2}$ x Nos. 7, 8, 9, and $\frac{3}{16}$ in................	1.30 “
$\frac{1}{2}$ x Nos. 10, 11, 12 and $\frac{1}{8}$ in..............	1.50 “
$\frac{7}{16}$ x Nos. 7, 8, 9 and $\frac{3}{16}$ in................	1.80 “
$\frac{7}{16}$ x Nos. 10, 11, 12 and $\frac{1}{8}$ in..............	2.10 “
$\frac{3}{8}$ x Nos. 7, 8, 9 and $\frac{3}{16}$ in................	1.90 “
$\frac{3}{8}$ x Nos. 10, 11, 12 and $\frac{1}{8}$ in..............	2.40 “

2. **Cost of Fabrication.**—The cost of fabrication of structural steel work may be divided into (1) cost of drafting; (2) cost of mill details; and (3) cost of shop labor.

Cost of Drafting.—The cost of drafting varies with the character of the work, and the tonnage to be made from one detail, so that costs per ton may mean but little. The amount of work on a standard sheet 24 in. × 36 in. is also a variable quantity. As a rough estimate the cost of standard sheets may be placed at from $10.00 to $15.00 per sheet.

Details of circular grain or ore bins cost from $1.50 to $3.00 per ton; details of conical or hopper bottoms cost from $4.00 to $6.00 per ton. Details of rectangular hoppered bins cost from $2.00 to $4.00 per ton, including columns and bracing.

Actual Cost of Drafting.—The cost for detailing 8 steel grain tanks, weight 700 tons, was $1.20 per ton, not including distribution for operating, which was estimated at $0.80 per ton, making total actual cost $2.00 per ton.

The cost for detailing 8 cylindrical tanks with spherical bottoms, including columns and bracing and weighing 100 tons, was $3.00 per ton, not including distribution for operating expenses, which was estimated at $2.00 per ton, making total actual cost $5.00 per ton.

For additional costs of detailing see "The Design of Steel Mill Buildings," Chapter XXVIII.

Cost of Mill Details.—The American Bridge's Co.'s card for cost of mill details differs somewhat from the standard card of cost of mill details given in the author's book on "The Design of Steel Mill Buildings," Chapter XXVIII.

American Bridge Co.'s card of cost of mill details:

Mill Rates:

"*a*"—0.15 cts. per lb.

This covers:

Plain punching 1 size hole in web only,

Plain punching 1 size hole in one or both flanges,

"*b*"—0.25 cts. per lb.

This covers plain punching one size hole in either web and one flange or web and both flanges. (The holes in the web and flange must be of the same size.)

"*c*"—0.30 cts. per lb.

This covers:

Punching of 2 size holes in the web only,

Punching of 2 size holes in one or both flanges.

"*d*"—0.35 cts. per lb.

This covers punching and assembling into girders. Coping, ordinary beveling, including riveting and bolting of standard connection angles (this class includes beams shipped with connection angles bolted).

"*e*"—0.40 cts. per lb.

This covers the punching of one size hole in the web and another size hole in the flanges.

"*f*"—0.15 cts. per lb.

This covers cutting to length with less variation than plus or minus $\frac{3}{8}''$.

"*r*" — 0.50 cts. per lb.

This covers beams with cover plates, shelf angles and ordinary riveted beam work, unless they are charged under class "*d*."

If this work consists of bending or any unusual work, the beams should not be included in the beam classification but estimated the same as riveted work. On all material estimated for cost at mill rates, 10 cts. per 100 lbs. is to be allowed for painting and 5 cts. per 100 lbs. is to be allowed for drawings.

Fittings.—All fittings, whether loose or attached, such as angle connections, bolts, separators, tie rods, etc., whenever they are estimated on in connection with beams or channels, to be charged at 1.55 cts. per lb. over and above the base price. The extra charge for painting is to be added to the price for fittings also. The base price on which fittings are based is not the base price of the beams to which they are attached, but is in all cases the base price of beams 15" and under. The above rates will not include painting or oiling, which should be charged at the rate of 0.10 cts. per lb. for one coat, over and above the base price plus the extra specified above. For plain punched beams, where holes of more than two sizes are used, 0.15 cts. per lb, should be added for each additional size hole; for example—Plain punched beams, where three size holes occur, would be indicated as "*e*" plus 0.15 cts.; four size holes as "*e*" plus 0.30 cts.; for example—A beam with ⅝" and ¾" holes in the flanges and ⅝" and ¾" holes in the web should be included in class "*e*."

Cutting to length can be combined with any of the other rates except "*d*" and would have to be indicated, for example—Plain punching one size hole in either web and one flange, or web and both flanges, and cutting to length would be marked "*bf*," which would establish a total charge of 0.40 cts. per lb.

Note to class "*d*":

No extra charge can be rendered to this class for punching various size holes or cutting to length; in other words, if a beam is coped, or has connection angles riveted or bolted to it, it makes no difference how many size holes are punched in this beam—the extra will always be the same, namely, 0.35 cts.

Beams with shelf angles, short seat angles or cover plates are strictly not covered by card rates. They can be charged either under class "*d*," this rate covering only the beam proper, in which case all other material ought to be rated as fittings with the charge of 1.55 cts. per lb. over and above the base price, or they can be classified under a special shop rate, "*r*,"—0.50 cts. per lb. This rate applies to all material forming the piece. It is the intention to charge whatever figures are the lowest, in order to give the customer the benefit of the doubt. In preparing the estimate, beam material should be marked with the letter "*b*" and to this should be added the letter giving the classification, thus: A beam punched with one size hole in one or both flanges will be marked "*b a*," etc.

In ordering material from the mill the following items should be borne in mind. Where beams butt at each end against some other member, order the beams ½ inch shorter than the figured lengths; this will allow a clearance of ¼ inch if all beams come ⅜ of an inch too long. Where beams are to be built into the wall, order them in full lengths making no allowance for clearance. Order small plates in multiple lengths. Irregular plates on which there will be considerable waste should be ordered cut to templet. Mills will not make reentrant cuts in plates. Allow ¼ of an inch for each milling for members that have to be faced. Order web plates for girders ¼ to ½ inch narrower than the distance back to back of angles. Order as nearly as possible every thing cut to required length, except where there is liable to be changes made, in which case order long lengths.

It is often possible to reduce the cost of mill details by having the mills do only part of the work, the rest being done in the field, or by sending out from the shop to be riveted on in the field connection angles and other small details that would cause the work to take a very much higher price. Standard connections should be used wherever possible, and special work should be avoided.

The classification of iron and steel bars is given in Table XXXII. The full extra charges for sizes other than those taking the base rate are seldom enforced; one-half card extras being very common.

In estimating the cost of plain material in a finished structure the shipping weight from the structural shop is wanted. The cost of material f. o. b. the shop must therefore include the cost of waste, paint material, and the freight from the mill to the shop. The waste is variable but as an average may be taken at 4 per cent. Paint material may be taken as one dollar per ton. The cost of plain material at the shop would be

Average cost per pound f. o. b. mill say	1.75	cts.
Add 4 per cent for waste	.07	"
Add $1.00 per ton for paint material	.05	"
Add freight from mill to shop (Pittsburg to St. Louis)	.225	"
Total cost per pound f. o. b. shop	2.095	"

To obtain the average cost of steel per pound multiply the pound price of each kind of material by the percentage that this kind of material is of the whole weight, the sum of the products will be the average pound price.

Shop Cost.—The following estimated shop costs include the cost of detailing and shop labor. The costs include an allowance of 25 per cent to cover distribution for contracting and administration. To obtain base costs subtract 20 per cent from the values given.

TABLE XXXIIa.

SHOP COST OF CIRCULAR AND RECTANGULAR BINS NOT INCLUDING HOPPERS OR BOTTOMS.

Thickness of Metal.	Shop Cost in Cents per Lb.
$\frac{1}{4}''$	0.80
$\frac{5}{16}$	0.75
$\frac{3}{8}$	0.70
$\frac{1}{2}$	0.65

TABLE XXXIIb.

SHOP COST OF BOTTOMS FOR CIRCULAR AND RECTANGULAR BINS.

Thickness of Material.	Flat Bottom Cents per Lb.	Spherical Bottom Cents per Lb.	Conical Bottom Cents per Lb.	Hopper Bottom Cents per Lb.
$\frac{1}{4}''$	1.50	4.00	3.50	2.50
$\frac{5}{16}$	1.45	4.15	3.00	2.40
$\frac{3}{8}$	1.40	4.40	2.75	2.25
$\frac{1}{2}$	1.25	4.50	2.50	2.00

For the shop cost of bracing, columns and trusses see "The Design of Steel Mill Buildings," Chapter XXVIII. (The cost of shop work has increased from 20 to 25 per cent since the shop costs in "Steel Mill Buildings" were prepared.)

3. **Cost of Erection.**—In estimating the cost of erection of bins it is best to divide the cost into (*a*) cost of placing and bolting the steel, and (*b*) cost of riveting. The cost will be based on labor at $3.00 per day of 8 hours.

(*a*) *Cost of Placing and Bolting.*—The cost of placing and bolting up plain grain tanks may be estimated at from $10.00 to $15.00 per ton. The cost of placing and bolting up circular or rectangular ore or coal bins with hopper bottoms may be estimated at from $12.00 to $18.00 per ton.

(*b*) *Cost of Riveting.*—It will cost from 6 to 10 cents per rivet to drive ⅝ or ¾ inch rivets by hand in structural framework where a few rivets are found in one place. A fair average is 7 cents per rivet. The same size rivets can be driven in tank work for from 4 to 7 cents per rivet, with 5 cents per rivet as a fair average.

The cost of riveting by hand is distributed about as follows:

3 men, 2 driving and 1 bucking up, at $3.50 per day of 8 hours	$10.50
1 rivet heater at $3.00 per day of 8 hours	3.00
Coal, tools, superintendence	1.50
Total per day	$15.00

On structural work a fair day's work driving ¾″ or ⅝″ rivets will be from 150 to 250, depending upon the amount of scaffolding required. This makes the total cost from 6 to 10 cents per rivet.

On bin work when the rivets are close together and little staging is required the gang above will drive from 200 to 400 rivets per day. This makes the total cost from about 4 to 7 cents per rivet.

Rivets can be driven by power riveters for one-half the above or less, not counting the cost of installation.

Soft iron rivets ½ inch and under can be driven cold for about one-half what the rivet can be driven hot, or even less.

Actual Erection Costs.—A steel grain elevator, including steel working house and 8 steel tanks and weighing 700 tons, cost $20.00 per ton to erect complete. This is a fair average for steel elevators.

A steel ore bin having 8 circular tanks with spherical bottoms and framework, weighing 100 tons, cost $20.00 per ton to erect and rivet complete. For additional costs of erecting grain elevators, see Chapter XX.

COST OF REINFORCED CONCRETE BINS.—The plans and details of reinforced concrete bins differ so much that it is difficult to give a general analysis. For the cost of concrete, forms, etc., see Chapter VII. The costs of several reinforced concrete bins are given in Chapter XII. For the cost of reinforced concrete grain bins, see Chapter XX. Reinforced concrete bins cost about the same as steel bins of the same capacity.

Relative Costs of Bins.—Suspension bunkers are cheaper than other types of bins, costing only 50 to 70 per cent of the cost of rectangular bins. Circular bins are slightly cheaper than rectangular bins. Reinforced concrete bins cost approximately the same as steel bins of the same type. Wooden bins cost about one-half as much as concrete or steel bins.

For the cost of grain bins see Chapter XX. For the cost of miscellaneous material and methods of calculating same see "The Design of Steel Mill Buildings," Chapter XXVIII.

Cost of Painting.—The amount of materials to make a gallon of paint and the surface covered by one gallon are given in Table XXXIII.

TABLE XXXIII.

Average Surface Covered per Gallon of Paint.

Paint.	Volume of Oil.	Pounds of Pigment.	Volume and Weight of Paint.	Square Feet.	
				1 Coat.	2 Coats.
			Gals. Lbs.		
Iron oxide (powdered)	1 gal.	8.00	1.2 = 16.00	600	350
Iron oxide (ground in oil)	1 "	24.75	2.6 = 32.75	630	375
Read lead (powdered)	1 "	22.40	1.4 = 30.40	630	375
White lead (ground in oil)	1 "	25.00	1.7 = 33.00	500	300
Graphite (ground in oil)	1 "	12.50	2.0 = 20.50	630	350
Black asphalt	1 " (turp.)	17.25	4.0 = 30.00	515	310
Linseed oil (no pigment)	1 "	......	....	875	......

Light structural work will average about 250 square feet, and heavy structural work about 150 square feet of surface per net ton of metal, while No. 20 corrugated steel has 2,400 square feet of surface.

It is the common practice to estimate ½ gallon of paint for the first coat and ⅜ gallon for the second coat per ton of structural steel, for average conditions.

The price of paint materials in small quantities in Chicago are (1906) about as follows: Linseed oil, 50 to 60 cents per gal.; iron oxide, 1 to 2 cents per lb.; red lead, 7 to 8 cents per lb.; white lead, 6 to 7 cents per lb.; graphite, 6 to 10 cents per lb.

A good painter should paint 1200 to 1500 square feet of plate surface or corrugated steel or 300 to 500 square feet of structural steel work in a day of 8 hours; the amount covered depending upon the amount of staging and the paint. A thick red lead mixed with 30 lbs. of lead to the gallon of oil will take fully twice as long to apply as a graphite paint or linseed oil.

For additional information on paints and painting see Chapter XI, also see "The Design of Steel Mill Buildings, Chapter XXVII.

CHAPTER XIV.

METHODS OF HANDLING MATERIALS.

Introduction.—The problem of handling material by mechanical means is a large and important one and it will be possible only to outline a few of the more common methods of handling coal, ashes, ore, grain, etc. Material is handled (*a*) continuously, or (*b*) intermittently. Material is handled continuously by means of elevators, steel screw conveyors, belt conveyors, push plate conveyors, traveling trough conveyors, and pneumatic conveyors. Material is handled intermittently by means of cable conveyors, and cars with rope haulage.

Elevators.—An appliance for lifting material from one level to another is called an elevator. Elevators for handling different materials necessarily have different details. The elevators shown in Figs. 127 to 129 are for handling grain. In Fig. 127 is shown an elevator leg with the boot for receiving the grain at the bottom and the elevator head for discharging the grain at the top. The buckets carrying the grain are commonly made of steel and are carried either on a chain or a flat belt usually of rubber.

Elevators for handling coal, ore, ashes, etc., are arranged in a somewhat different manner from those for handling grain. The elevator and conveyor shown in Fig. 130 was designed by the Link Belt Machinery Co. to convey and elevate coal. The coal is fed onto the conveyor by an automatic feeder. This type of elevator is much used in coaling plants.

A chain belt elevator for elevating coal into a circular steel storage bin placed outside the power house is shown in Fig. 131.

A railway coaling station is shown in Fig. 132. In this station the coal is dumped or shoveled from cars and is elevated by chain belt elevators to the coal pockets.

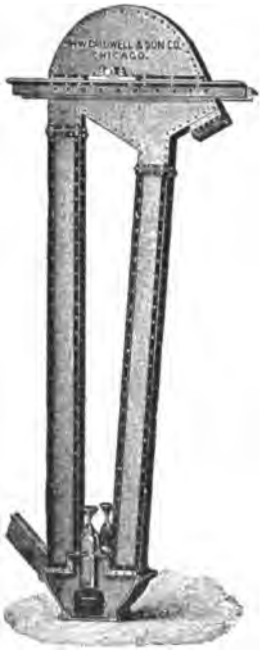


FIG. 127.
ELEVATOR LEG.

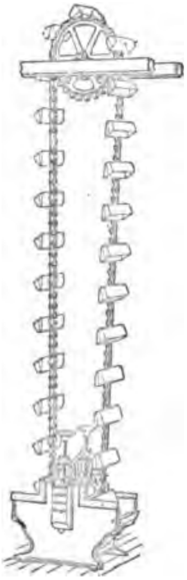


FIG. 128.
CHAIN ELEVATOR.

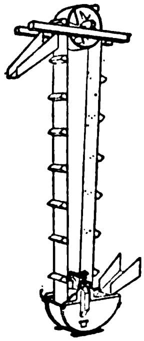


FIG. 129.
BELT ELEVATOR.

A typical arrangement of elevators, bins and spouts for a power plant, as designed by the Jeffrey Manufacturing Co., for the Cleveland Arcade Building, Cleveland, Ohio, is shown in Fig. 133. This eleva-

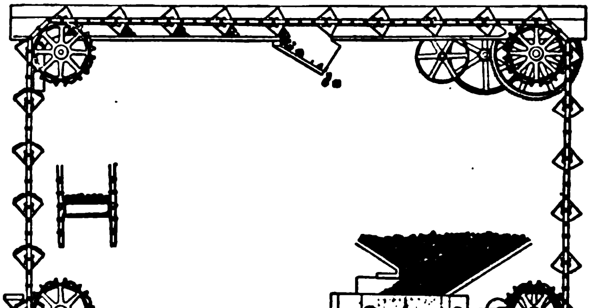


FIG. 130. LINK BELT ELEVATOR AND CONVEYOR.

tor handles 30 tons of coal per hour. Ashes and coal should not be handled by the same elevator for the reason that while coal protects the metal, ashes corrode the parts and choke the elevator.

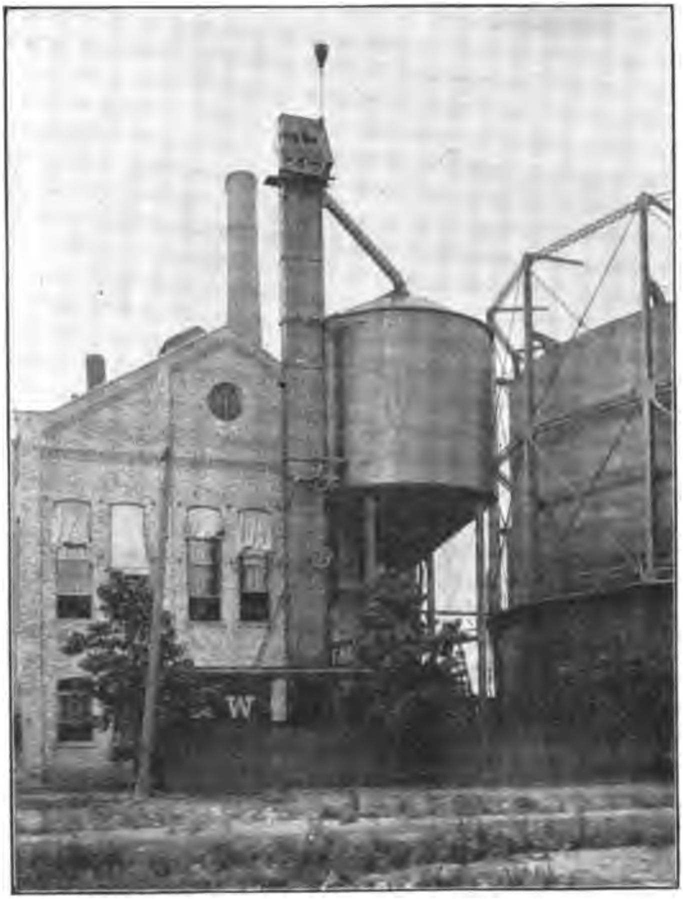


FIG. 131.

Steel Screw Conveyors.—Steel screw or worm conveyors have been extensively used for conveying grain, flour, and similar materials for comparatively short distances. The continuous spiral conveyor is most often used, although conveyors with paddles are used for mixing and

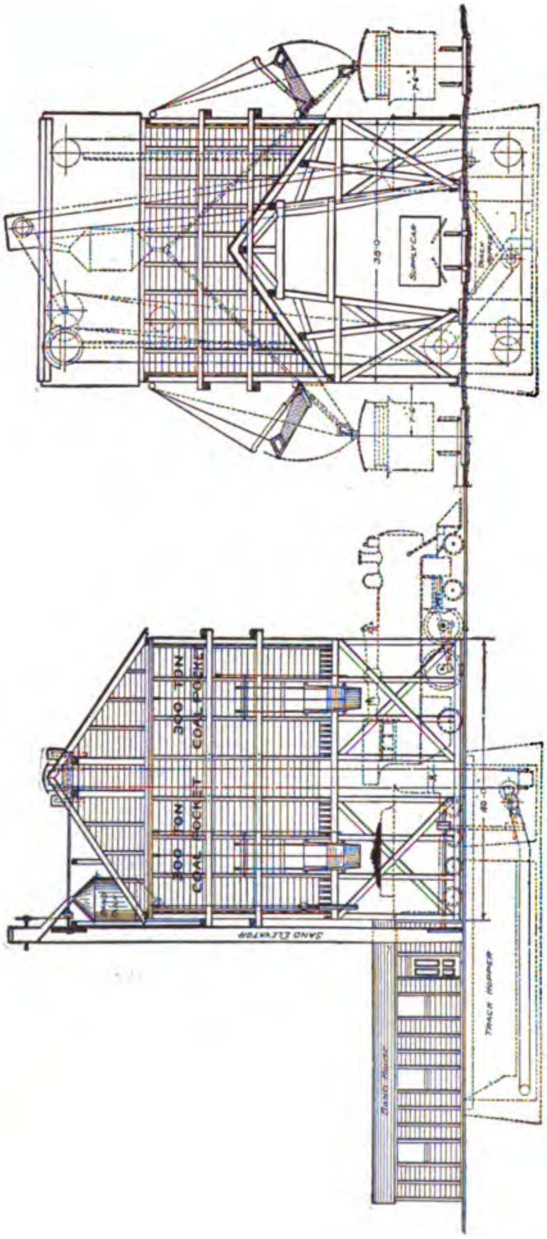


FIG. 132. COALING STATION DESIGNED BY JEFFREY MANUFACTURING CO.

conveying materials. The conveyor hangers require care to insure clear action. The conveyor should not be driven too fast or the material will be carried around without being advanced. Details of screw conveyors are shown in Fig. 134.

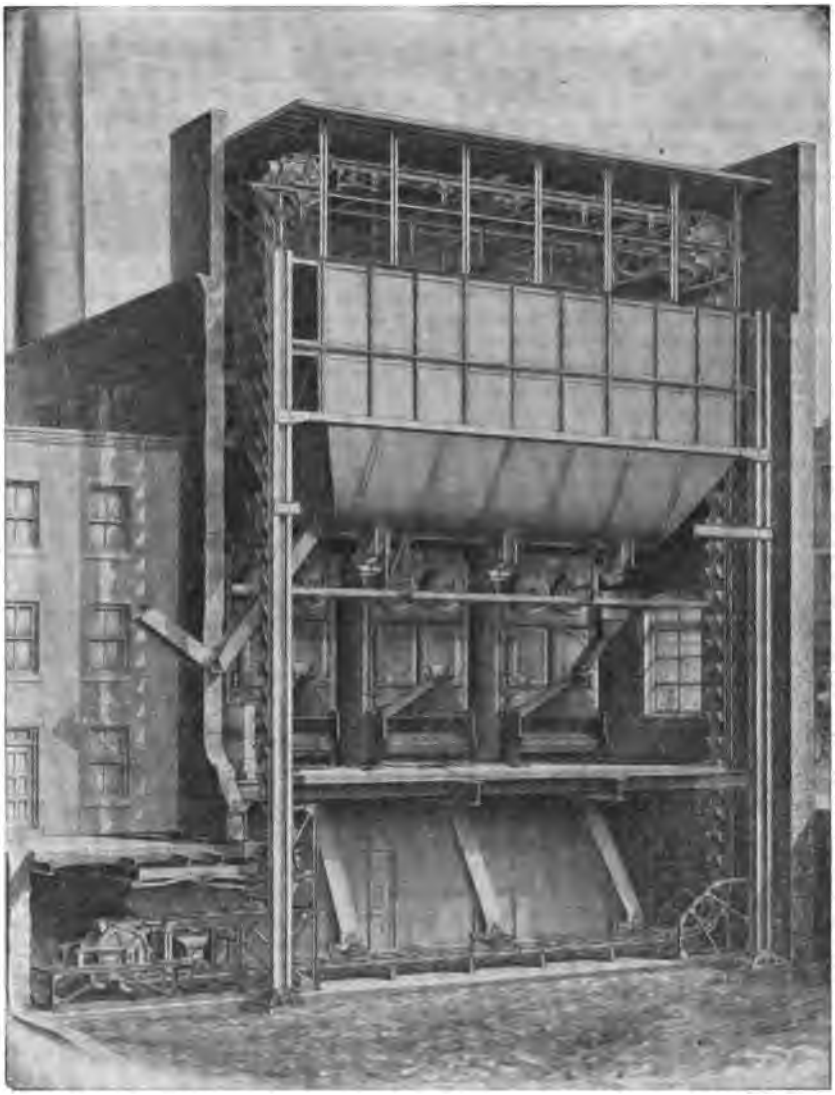
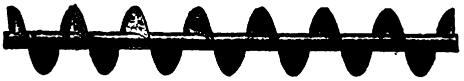


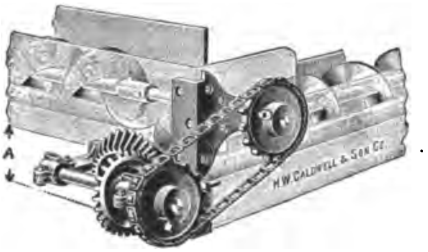
FIG. 133. TYPICAL COAL AND ASH HANDLING PLANT.

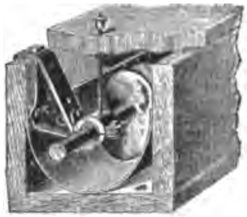


STEEL CONVEYOR

H.W. CALDWELL & SON CO CHICAGO

DELIVERY GATE





RIGHT-ANGLE CONVEYOR DRIVE. CONVEYOR HANGER

FIG. 134.

The following experiments on a conveyor 6 inches in diameter, with 2 inches pitch and a one-inch shaft are recorded in Zimmer's "Mechanical Handling of Materials."

Revolutions per Minute.	Material Delivered in Cu. Ft. per Minute.
60	28
80	36
100	30
140	Delivery ceased altogether.

The principal advantages of screw conveyors are their simplicity and low first cost. Screw conveyors are extensively used in country grain elevators, while belt conveyors are commonly used for large grain elevators.

Belt Conveyors.—Belt conveyors are used chiefly for conveying grain and other material in a finely divided state, although coarse coal and ore may be handled very satisfactorily if the conveyors are properly designed. The belts may be run flat as in Fig. 135, or trough-shaped as in Fig. 136. It has been found that grain will be carried on flat belts if the material is properly delivered to the belt at the same speed as the belt. Where flat belts are used the belt is usually hollowed at the point where the material is fed onto the conveyor. To deliver the material at any point the direction of the belt is suddenly changed downward by passing over a pulley on the throw-off carriage. The grain leaves the belt by its own momentum and falls into a hopper. Belts are kept properly tightened by an automatic belt tightener. Belt conveyors for handling coal, coke, etc., are always run on grooved rollers as shown in Fig. 136. Conveyor belts for handling grain are commonly made of rubber reinforced with cotton warp. Canvas belts are also used for grain conveyors. A conveyor gallery of a grain eleva-

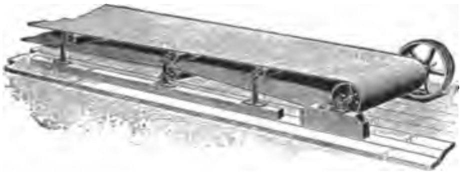


FIG. 135.

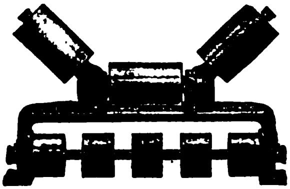


FIG. 136.

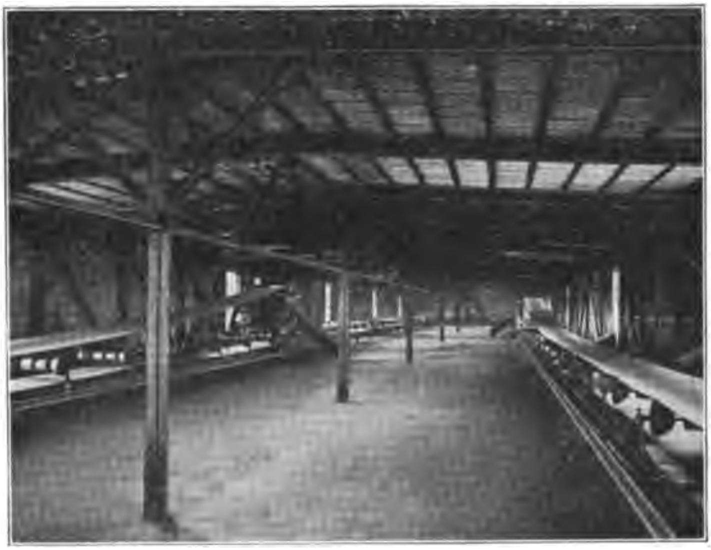


FIG. 137. CONVEYOR GALLERY—ALBERT DICKINSON CO.'S ELEVATOR, MINNEAPOLIS, MINN.

tor is shown in Fig. 137. This conveyor gallery contains two belt conveyors that carry grain from the working house to the steel grain storage bins.

Conveyor belts for handling coal, coke, etc., are made stronger than for grain and are sometimes made of woven wire for handling washed coal, etc.

Belt conveyors may be operated on a maximum slope of 17 to 24°, depending upon the material and the belt. The maximum limit of 17° is commonly used. The maximum speed of belt conveyors for handling oats and other light grains should not exceed 400 feet per minute, while for wheat the belt may be run at a speed of 600 feet per minute.

Belt conveyors carrying large pieces of coal, coke, etc., should be run slowly, 150 to 250 feet per minute, while for finer materials the belt may be run as high as for wheat.

Belt conveyors are expensive to install and require careful supervision. The use of belt conveyors in large grain elevators is almost universal. A belt conveyor is used for handling coal in the power plant in Fig. 138.

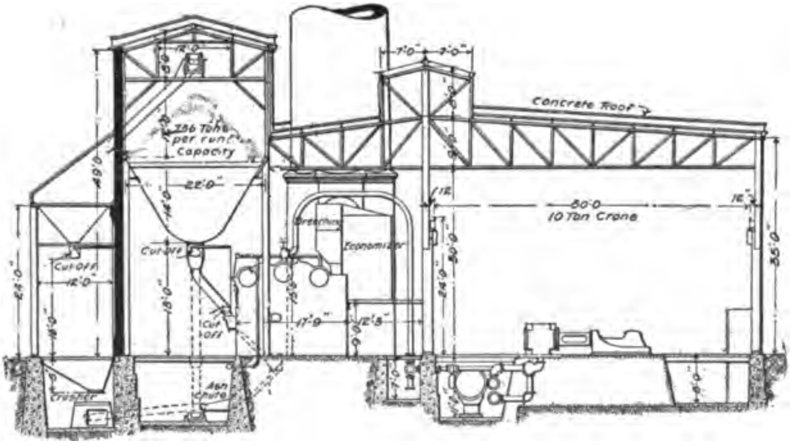


FIG. 138. TYPICAL ARRANGEMENT OF A COAL AND ASH HANDLING PLANT.

Push Plate Conveyors.—Conveyors in which the material is pushed along in metal or wooden troughs are used for handling coal, coke, etc.

Conveyors of this type should never be used for gritting materials, but sticking material can be handled in this manner much more satisfactorily than by means of the screw conveyor. The Link Belt elevator shown in Fig. 130 is a push plate conveyor and a chain elevator combined.

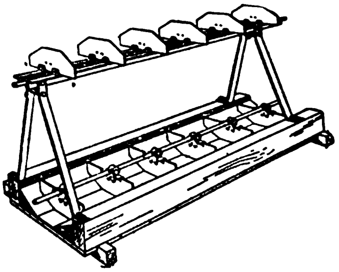


FIG. 139 a.

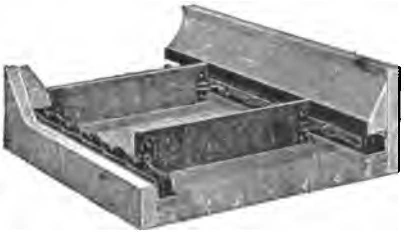


FIG. 139 b.

A push plate conveyor with metal plates and a bar chain is shown in Fig. 139 a, and a push plate conveyor with wooden plates and a chain is shown in Fig. 139 b.

Traveling Trough Conveyors.—The traveling trough conveyor is simply a metal band conveyor with channel sides. The operation is clearly shown in Fig. 140. This conveyor has a movable unloader which makes it possible to dump at any point. This type of conveyor is much used in handling coal.

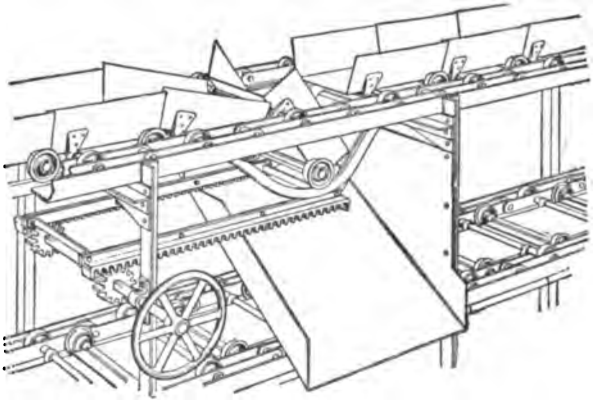


FIG. 140.

Pneumatic Conveyors.—Pneumatic conveyors are used for conveying and elevating grain. Pneumatic conveyors are of three types: (1) the blast system; (2) the suction system; and (3) the combined blast and suction system.

The pneumatic system will be illustrated by describing the operation of the system, Fig. 141, in use by the Steel Storage and Elevator Construction Co., Buffalo, N. Y., which is of the combined blast and suction type.

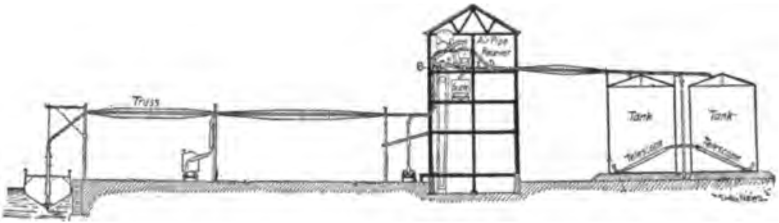


FIG. 141.

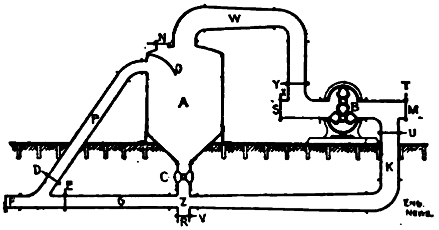


FIG. 142.

Fig. 142 shows the main operating device located in the elevator house In this device *A* is an air-tight receiver with a hopper bottom, and openings as shown. Underneath the receiver is a continuous feed device, *C*, which is operated by means of a belt from a pulley on the line shaft. The blast pipe *K*, leading to the point of discharge under the receiver, is made tapering to increase the current of air created by the blower *B*, which always runs in the same direction.

To convey grain from the elevator to the storage tanks, grain is admitted into the receiver through valve *N*. Valves *X*, *U*, and *E* are opened, and valves *T*, *V*, *Y* and *D* are closed. The grain passes through the feed device *C*, comes in contact with the blast and is forced through the pipes *G* and *F* to the storage tanks, as shown in Fig. 141.

To transfer grain from the storage tanks, valves *D, T, V, Y* and *M* are opened, and valves *E, I, U* and *X* are closed, which changes the air blast to suction. Grain is drawn from the bins by suction entering receiver *A* through pipe *P,* and can be drawn off through opening *R* to stock bins, etc.

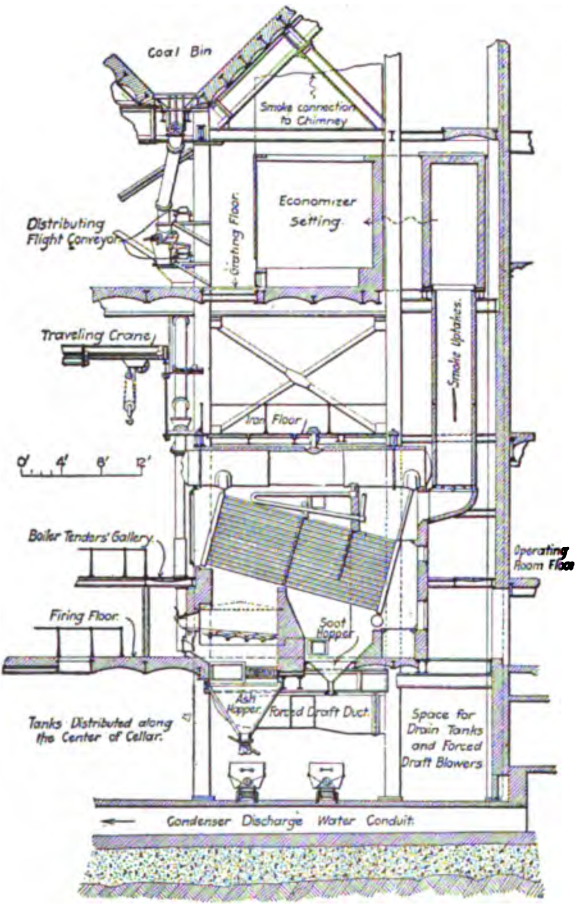


FIG. 143.

The principal objection to the pneumatic system for handling grain is the large initial cost and large power cost for operation.

Cable Conveyors.—In cable conveyors the buckets conveying the material are attached to a trolley which is carried on a fixed rope and is moved along by a runner, or the buckets are attached directly to a moving cable or runner which is carried on pulleys. Cable conveyors are much used where it is difficult to erect and operate a car system and for coaling at sea.

Car with Rope Haulage.—This method is used where the material is to be moved a considerable distance, or where an intermittent system is preferred. Cars are made self dumping, and may have a capacity of from ½ to 3 or 4 tons.

An example of the use of this system for handling ashes is given in Fig. 143.

Miscellaneous Methods.—Coal is taken from vessels by means of clam shell buckets, which are lowered into the vessel, are lifted, carried into the storage shed by means of a trolley and dumped. There are many variations of this method in use by different companies. Coal, ashes, etc., are sometimes elevated in cars on a cage or in a skip as in mines.

Coal is sometimes loaded on cars from stock piles by means of steam shovels. Coal and ore may be unloaded by dumping through the bottom in dump cars, or box cars are lifted and turned upside down.

PART III.

THE DESIGN OF GRAIN BINS AND ELEVATORS.

Introduction.—The grain elevator plays a very important part in the handling of grain in America. The grain (wheat, corn, oats, rye, etc.) is hauled in wagons to the country elevator at the railroad station. The wagon is weighed and driven up an incline onto the dump; here the back end of the wagon is dropped and the front end raised so that the grain runs into the dump hopper, the empty wagon is then weighed and the amount of grain determined. From the dump the grain runs by gravity into the boot of the elevator leg. The elevator leg consists essentially of two pulleys, one in the boot, and the other at the head, over which runs a belt, either a rubber or a chain belt; at intervals along the belt are metal buckets attached, which scoop up the grain from the boot and carry it to the head, where it is discharged into spouts, which deliver the grain onto conveyor belts which in turn carry the grain to other points, or discharge directly into bins. The grain in the country elevator is elevated by the elevator leg and conveyed or spouted into bins. If it is to be stored for some time, it may be conveyed to storage bins outside the main elevator or working house by a steel screw or belt conveyor. Usually it will be run through a cleaner where the dust and defective grains are removed, then it is elevated into the shipping scale hopper, where it is weighed and run directly into the cars by gravity.

When the cars loaded with grain arrive at a terminal elevator, where the grain is to be reshipped by rail or water, they are delivered to the elevator, and are placed alongside hoppers near the receiving legs by means of a car puller. These hoppers are either in the working house or in a shed annex alongside. The cars are unloaded by two power car shovels, each shovel being operated by one man, who carries a large scoop at the end of the shovel rope back into the car. When he stops and allows the rope to slack, the mechanism of the spool pulls the scoop loaded with grain to the car door where it stops automatically. The grain falls into the hopper and runs by gravity, or is conveyed by belt conveyors to the boot of the receiving elevator leg, where it is carried by the elevator buckets to the top of the leg, and discharged into garner hoppers mounted on scales, where it is weighed. It is then conveyed by conveyor belts to the bin it is to occupy, or is run into the cleaner bin and through the cleaner, and then is conveyed to the proper bin.

Bins are emptied through spouts in the bottom directly into cars, or the grain is conveyed to the boot of the shipping elevator leg, which elevates it to the shipping hopper bins mounted on scales, from which it is spouted directly into cars or is conveyed to other bins.

In many elevators the grain is cleaned, polished, dried, and graded to improve its quality.

When grain is received directly into the elevator from vessels a "marine" leg is used. This is equipped with a mechanism for raising or lowering, so that the grain may be elevated from the hold of the vessel into the building.

In filling vessels, the grain may be spouted directly from the bins, as is done on the Great Lakes, or, as is common for ocean vessels, may be conveyed by conveyor belts through shipping galleries, from which it is spouted into the vessel. This makes it possible to load several vessels at the same time. In this connection it should be remembered that lake vessels carry full cargoes of grain, while seagoing vessels usually carry mixed cargoes.

The foregoing description will give an idea of the function of the grain elevator. Grain elevators are of several different types, which will now be described.

CHAPTER XV.

TYPES OF GRAIN ELEVATORS.

Introduction.—Grain elevators, or "silos," as they are called in Europe, may be divided into two classes according to the arrangement of the bins and elevating machinery: (*a*) elevators which are self contained, with all the storage bins in the main elevator or working house, as for example, the Great Northern Ry. steel elevator, West Superior, Wis.; and (*b*) elevators having a working house containing the elevating machinery, while the storage is in bins connected with the working house by conveyors, as for example, the Independent Elevator, Omaha, Neb. The working house is usually rectangular in shape, with square or circular bins; while the independent storage bins are usually circular.

With reference to the materials of which they are constructed, elevators may be divided into (1) timber; (2) steel; (3) concrete; (4) tile; and (5) brick.

1. **Timber.**—Timber was formerly the principal material used in building grain elevators, and is at present used very extensively for small country elevators. For large bins, the "crib" construction is most used. In this construction, pieces of 2″ × 4″, 2″ × 6″, or 2″ × 8″ are laid flatwise, so as to break joints and bind the structure together, and are spiked firmly. This makes a strong form of construction, and one very cheap with the former low price of lumber. Care must be used in filling elevators of this type for the first time, to fill all bins uniformly to prevent unequal settlement caused by the compression of the timber and the closing up of the horizontal joints. Cribbed timber bins have been known to settle 18 inches in a height of 70 feet. Timber elevators are liable to be destroyed by fire, and call for a very high rate of insurance. A view of a timber grain elevator during erection

is shown in Fig. 144. This elevator was designed and built by the John S. Metcalf Co., Chicago, Ill.

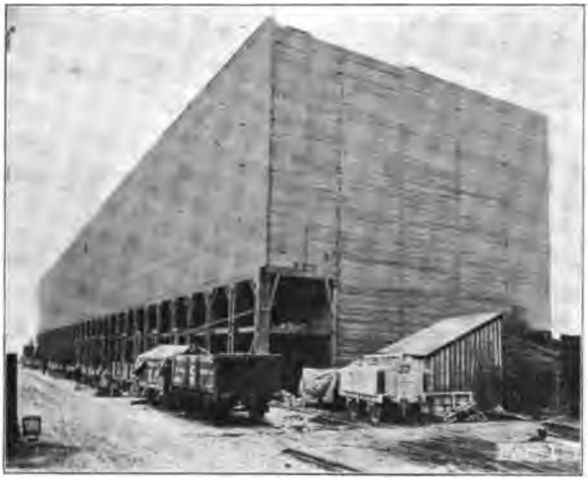


FIG. 144. TIMBER CRIBBED BIN, MANCHESTER SHIP CANAL CO.'S ELEVATOR NO. 7, MANCHESTER, ENGLAND.

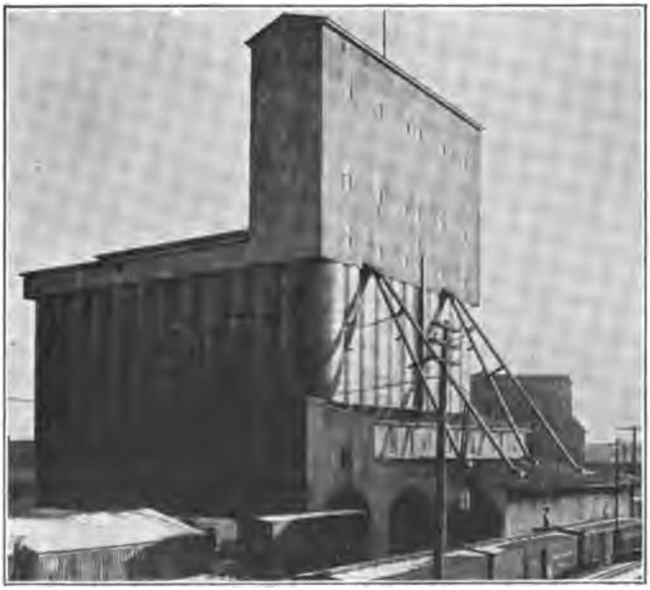


FIG. 145. L. S. AND M. S. RY. STEEL ELEVATOR, BUFFALO, N. Y.

2. **Steel.**—Steel has come rapidly into use in the last ten years for fire-proof elevator construction. The first steel elevator in the United States was the Washington Avenue elevator of the Grand Point Storage Company, Philadelphia, started in 1859 and completed in 1866. The 88 iron grain bins were 11 feet in diameter, and 45 feet high. After 37 years the iron was in an excellent state of preservation, the outside of the bins having been painted once in 7 years, and the inside not at all. Steel elevators are built (*a*) of the working house type with square bins, as in the Great Northern Ry. elevator, West

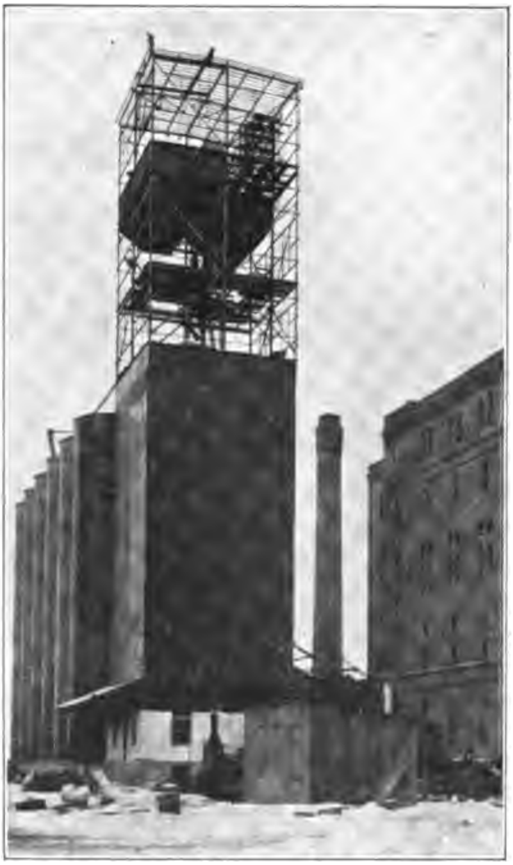


FIG. 146. THE WINONA MALTING CO.'S ELEVATOR UNDER CONSTRUCTION.

Superior, Wis., or the Windmill Point Elevator, Figs. 192 and 193, or with circular bins, as in the Buffalo Steel Elevator, or the L. S. & M. S. Ry. Steel Elevator in Fig. 145; or (*b*) of the working house and storage bin type, as in the Independent Elevator, Omaha, Neb., Chapter XIX, or the Winona Malting Co.'s steel elevator, Figs. 146 and 147.

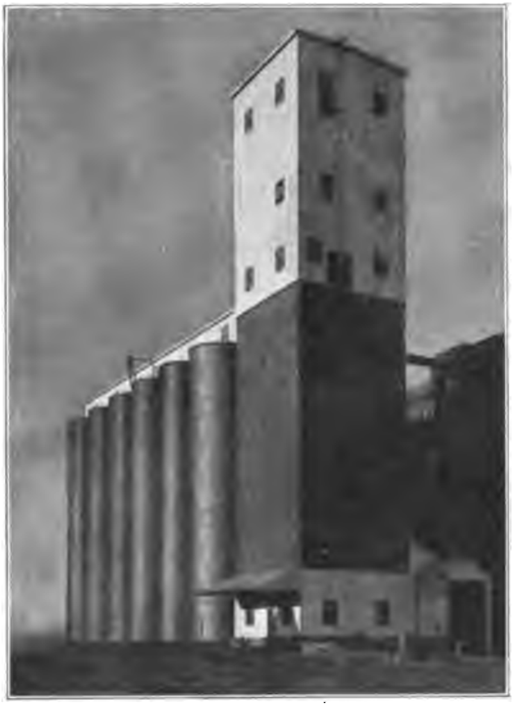


FIG. 147. STEEL GRAIN ELEVATOR FOR THE WINONA MALTING CO. OF WINONA, MINN.

(*a*) *Working House Steel Elevator.*—The "Great Northern" Steel Elevator, designed by Max Tolz, for the Great Northern Elevator Co., Buffalo, N. Y., in 1897, was one of the first notable examples of this type of elevator, while the Great Northern Elevator at West Superior, Wis., designed by Mr. Tolz, is an excellent example of a modern steel elevator of the working house type. These elevators are described in Chapter XVIII and Chapter XIX, respectively.

The L. S. & M. S. Steel Elevator shown in Fig. 145, constructed on the McDonald system, as described in Chapter XVIII; and the Windmill Point Steel Elevator, constructed on the Metcalf system, shown in Figs. 192 and 193, and described in Chapter XVIII, are excellent examples of this type of construction.

(*b*) *Working House and Steel Storage Bins.*—The Independent Elevator at Omaha, described in Chapter XIX, is an excellent example of an elevator of this type. The elevator shown in Fig. 146 and Fig. 147 is an excellent example of a small elevator of this type. The working house is 32′ 0″ × 32′ 0″ × 150′ 0″ high; and the bins are 20′ 0″ × 80′ 0″. The capacity of the working house is 55,000 bushels, of the bins 295,000 bushels, or a total capacity of 350,000 bushels. The working house cost about $1.00, and the bins about 14 cents per bushel of storage capacity. This elevator was designed and erected by the Minneapolis Steel & Machinery Co., Minneapolis, Minn. This type of elevator costs more per bushel for the working house, but very much less per bushel of storage, than grain elevators of the working house type (*a*).

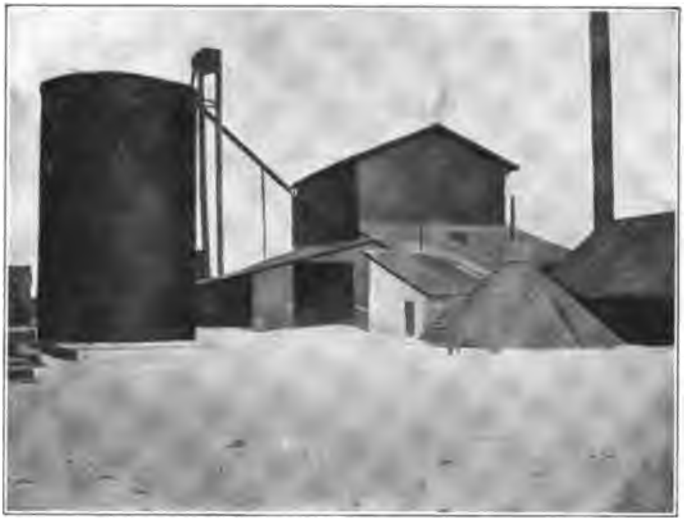


FIG. 147a. STEEL STORAGE BIN WITH ELEVATOR LEG.

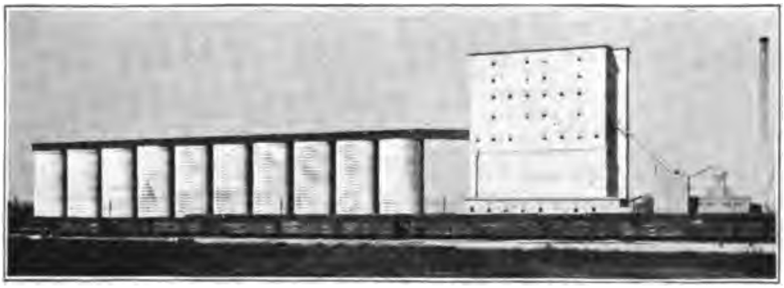


FIG. 148. CONCRETE GRAIN BINS, MISSOURI PACIFIC RY., KANSAS CITY, MO.

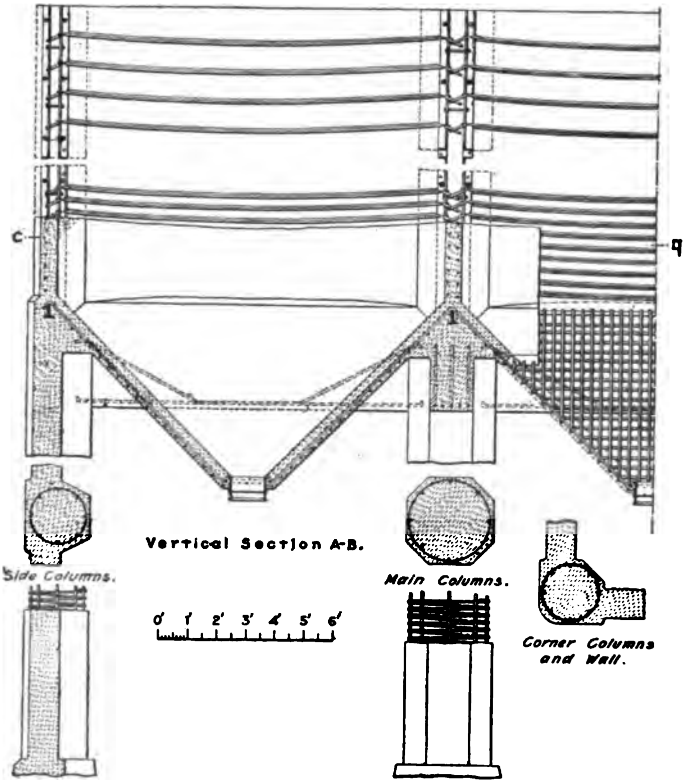
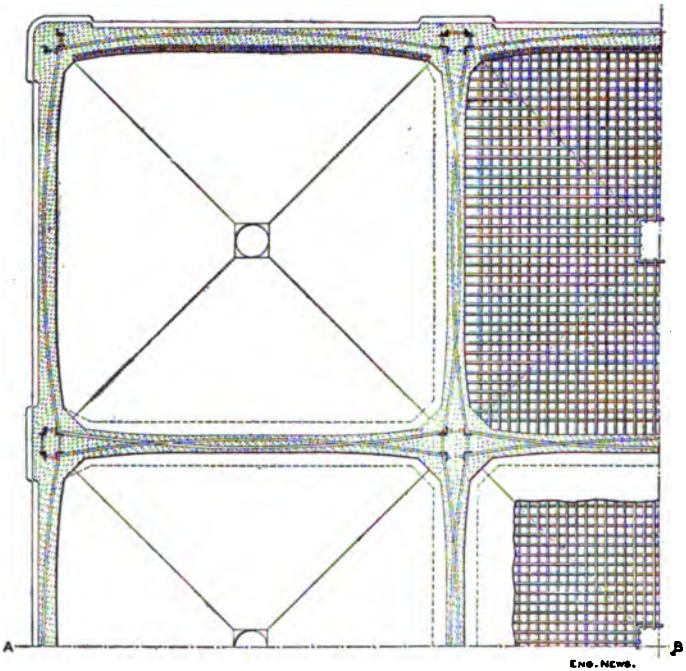


FIG. 149. VERTICAL SECTION OF REINFORCED CONCRETE GRAIN ELEVATOR DESIGNED BY J. A. JAMIESON.

A steel grain storage bin with a steel elevator leg outside of the bin is shown in Fig. 147 a. The elevator leg is arranged to fill the bin, or to raise the grain so that it can be spouted into a car or into other bins.

3. **Concrete.**—Reinforced concrete elevators are commonly built of type (*b*) with working house and storage bins. The working house is usually built of steel or timber, and the bins of reinforced concrete. The Missouri Pacific Ry. Co.'s reinforced concrete elevator is shown in Fig. 148. Specifications and a detailed description of this elevator are given in Chapter XVIII. The Santa Fe Elevator, Chicago, Ill., and the Canadian Pacific Elevator, Port Arthur, Ontario, are excellent examples of the use of reinforced concrete for storage bins, and are described in Chapter XIX.



Part Sectional Plan through C-D.

FIG. 150. HORIZONTAL SECTION OF REINFORCED CONCRETE GRAIN ELEVATOR DESIGNED BY J. A. JAMIESON.

The grain elevator shown in Fig. 149 and Fig. 150 was designed by Mr. J. A. Jamieson, Montreal, Canada. The bins are rectangular, and are carried on reinforced concrete columns, which continue up through the bin corners. In this design the interior reinforced concrete walls may be replaced by steel trough plate construction.

4. **Tile.**—Bins for storing grain are frequently made of tile construction reinforced with steel. There are two systems, both patented, one by the Barnett-Record Co., Minneapolis, Minn., and the other by the Witherspoon-Englar Co., Chicago, Ill. A bin in process of construction is shown in Fig. 151. The specifications for the construction of these bins are given in Chapter XVIII, and details of construction are given in Fig. 201.

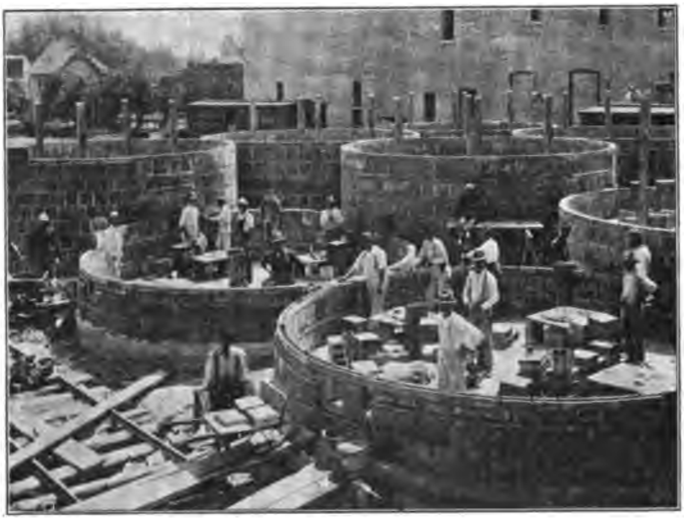


FIG. 151. TILE GRAIN BINS DURING CONSTRUCTION.

The Canadian Northern Ry. elevator, described in Chapter XIX, is an excellent example of tile bin construction, and was built by the Barnett-Record Co.

In Fig. 202 is shown the system patented by the Witherspoon-Englar Co., Chicago, Ill. This system is very much like the Barnett-Record system and is constructed in the same manner.

5. **Brick.**—*Circular storage bins* are constructed of brick in a manner similar to the tile construction described above. The following

description of a circular brick bin, 31′ 6″ in diameter and 80′ 0″ high, built by the Cleveland Elevator Building Co., Minneapolis, Minn., will illustrate the construction.

The bin wall is a three-course wall, the two inner courses being laid solid, a space of 3 inches being left between the two inner courses and the outer course, making a wall 16 inches thick. The outer course is bonded to the two inner courses by wire bonds, passing through the air space. A channel is formed every 12 inches in height in the inner wall by splitting a course of brick, making the channel about 2½ inches wide and a brick high, and a brick from the inner surface. The steel reinforcement is placed in this channel, and is grouted with one to two Portland cement grout, as in the construction of the tile bins.

Rectangular bins are made of brick as follows: The bins are made rectangular in sets, with brick pilasters at the outside corners, and columns at the interior corners of the bins. Bars passing through the pilasters and columns are then placed in the planes of the walls, and the walls are made of brick arches, with the concave side of the arch outside. This system works very well for fire-proof country elevators.

Comparison of Different Types.—The different types of fire-proof grain bins appear to fill all the requirements, and the question of local cost or preference will usually determine the type to select. The grain keeps well and is protected from weevil in all types. It has been claimed by some that steel bins sweat or heat the grain, but this is now an exploded theory. It is, however, a well known fact that green or damaged grain will heat and go through a sweat, whether contained in a timber or other type of bin. When heated the grain is changed from one bin to another and aired in transit. This makes it necessary to have an efficient conveyor system and an ample number of bins.

The cost of the different types of fire-proof bins is a matter of local conditions, the cost usually being about the same, although steel probably has a slight advantage over the other materials. For a detailed discussion of the cost of grain bins and elevators, see Chapter XX.

The working house in fire-proof elevators is generally made of steel frame construction, the working house of the Independent Elevator, described in Chapter XIX, being an excellent example.

The price of timber is advancing very rapidly, and this, together with the high insurance rates on timber elevators, will tend to limit the use of timber grain elevators.

CHAPTER XVI.

STRESSES IN GRAIN BINS.

Introduction.—The problem of calculating the pressure of grain on bin walls is somewhat similar to the problem of the retaining wall, but is not so simple. The theory of Rankine will apply in the case of shallow bins with smooth walls where the plane of rupture cuts the grain surface, but will not apply to deeper bins or bins with rough walls. (It should be remembered that Rankine assumes a granular mass of unlimited extent.)

Two solutions of the problem of the calculation of the pressure in grain bins have been proposed: (1) Janssen's Solution, and (2) Airy's Solution. The results by the two methods agree very closely with experiments.

Nomenclature:

The following nomenclature will be used:

$\phi =$ the angle of repose of the grain.
$\phi' =$ the angle of friction of the grain on the bin walls.
$\mu = \tan \phi =$ coefficient of friction of grain on grain.
$\mu' = \tan \phi' =$ coefficient of friction of grain on the bin walls.
$x =$ angle of rupture.
$w =$ weight of grain per cubic foot.
$V =$ vertical pressure of the grain.
$L =$ lateral pressure of the grain.

JANSSEN'S SOLUTION.—The bin in (*a*) Fig. 152, has a uniform area A, a constant circumference U, and is filled with grain weighing w per unit of volume, and having an angle of repose ϕ. Let V be the vertical pressure, and L be the lateral pressure at any point, both V and L being assumed as constant for all points on the horizontal plane. (More correctly V and L will be constant on the surface of a dome as in (*b*).)

The weight of the grain between the sections of y and $y+dy = A.w.dy$; the total frictional force acting upwards at the circumference will be $= L.U.\tan\phi'.dy$; the total perpendicular pressure on the upper surface will be $= V.A$; and the total pressure on the lower surface will be $= (V+dV)A$.

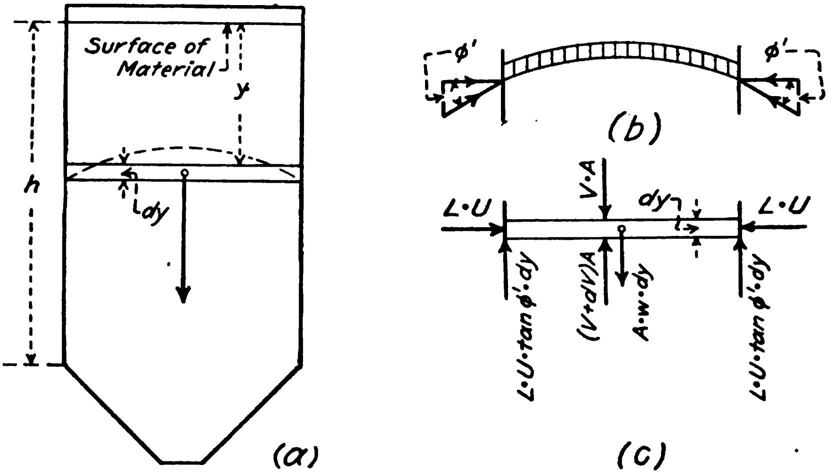


FIG. 152.

Now these vertical pressures are in equilibrium, and

$$V.A - (V+dV)A + A.w.dy - L.U.\tan\phi'.dy = 0,$$

and

$$dV = (w - L.\tan\phi'\frac{U}{A})dy \qquad (138)$$

Now in a granular mass, the lateral pressure at any point is equal to the vertical pressure times k, a constant for the particular grain, and

$$L = k.V$$

Also let $\frac{A}{U} = R$ (the hydraulic radius), and $\tan\phi' = \mu'$.

Substituting the above in (138) we have

$$dV = (w - \frac{k.V}{R}\tan\phi')dy$$

Now let

$$\frac{k.\mu'}{R}=n \tag{139}$$

and

$$\frac{dV}{w-n.V}=dy \tag{140}$$

Integrating (140) we have

$$\log(w-n.V)=-n.y+C \tag{141}$$

Now if $y=0$, then $V=0$, and $C=\log w$, and (141) reduces to

$$\log\left(\frac{w-n.V}{w}\right)=-n.y,$$

and

$$\frac{w-n.V}{w}=\frac{1}{e^{n.y}}=e^{-n.y},$$

where e is the base of the Naperian system of logarithms. Solving for V, we have

$$V=\frac{w}{n}(1-e^{-n.y}) \tag{142}$$

Substituting the value of n from (139), we have

$$V=\frac{w.R}{k.\mu'}\left(1-e^{-\frac{k.\mu'.y}{R}}\right) \tag{143}$$

Now if h be taken as the depth of the grain at any point, we will have

$$V=\frac{w.R}{k.\mu'}\left(1-e^{-\frac{k.\mu'.h}{R}}\right) \tag{144}$$

Also since

$$L=k.V,$$

$$L=\frac{w.R}{\mu'}\left(1-e^{-\frac{k.\mu'.h}{R}}\right) \tag{145}$$

Now if w is taken in lbs. per cu. ft., and R in feet, the pressure will be given in lbs. per square foot.

Now both μ' and k can only be determined by experiment on the particular grain and kind of bin. For wheat and a wooden bin, Janssen found $\mu'=0.3$ and $k=0.67$, making $k.\mu'=0.20$.

Jamieson found by experiment that for wheat $k=0.6$, and he found the following values for μ' with wheat weighing 50 lbs. per cu. ft. and having $\phi=28°$, $\mu=0.532$:

Wheat on steel trough plate bin,	$\mu' = 0.468$
Wheat on steel flat plate, riveted and tie bars,	$\mu' = 0.375$ to 0.400
Wheat on steel cylinder, riveted,	$\mu' = 0.365$ to 0.375
Wheat on cement-concrete, smooth to rough,	$\mu' = 0.400$ to 0.425
Wheat on tile or brick, smooth to rough,	$\mu' = 0.400$ to 0.425
Wheat on cribbed wooden bin,	$\mu' = 0.420$ to 0.450

For additional values of μ' for different grains and different bin walls see tests by Airy and Pleissner in Tables XXXV and XXXVII, Chapter XVII.

Experiments by Bovey, Lufft and the author would appear to prove that k may be very much less than the values given by Janssen and

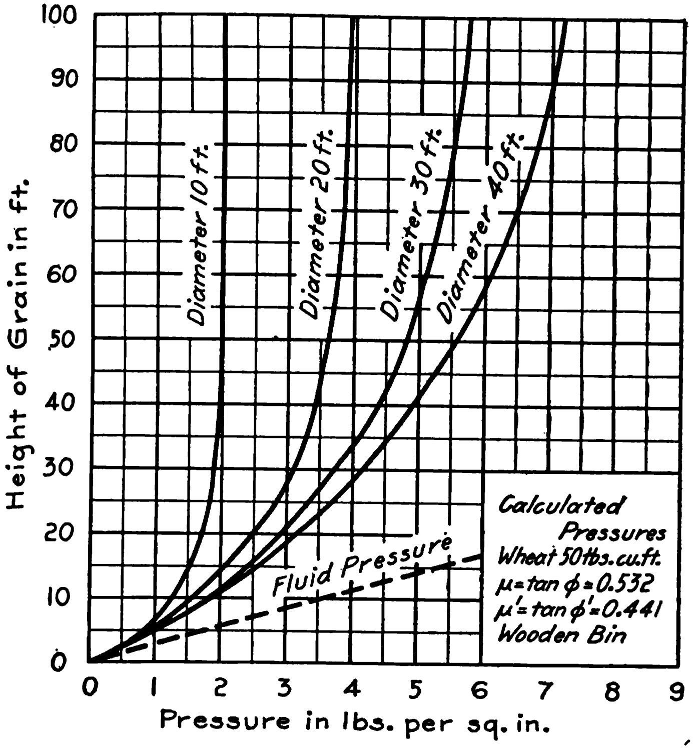


FIG. 153. LATERAL PRESSURE IN WOODEN GRAIN BINS CALCULATED BY JANSSEN'S FORMULA.

Jamieson, and that for wheat *k* varies from 0.3 to 0.6. Pleissner found that *k* varies with the grain and the bin walls (see Chapter XVII). However, it will be seen in (145) that the maximum lateral pressure in a bin which must be used in design of deep bins, is independent of *k*, and that therefore an exact determination of *k* is not very important. In calculating the values of *V* and *L* in (144) and (145), it is necessary to use a table of natural or hyperbolic logarithms. A brief table of hyperbolic logarithms is given in Table XXXIV. To find the hyperbolic logarithm of any number, use the relation: *The hyperbolic or Naperian logarithm of any number = common or Brigg's logarithm* × 2.30259.

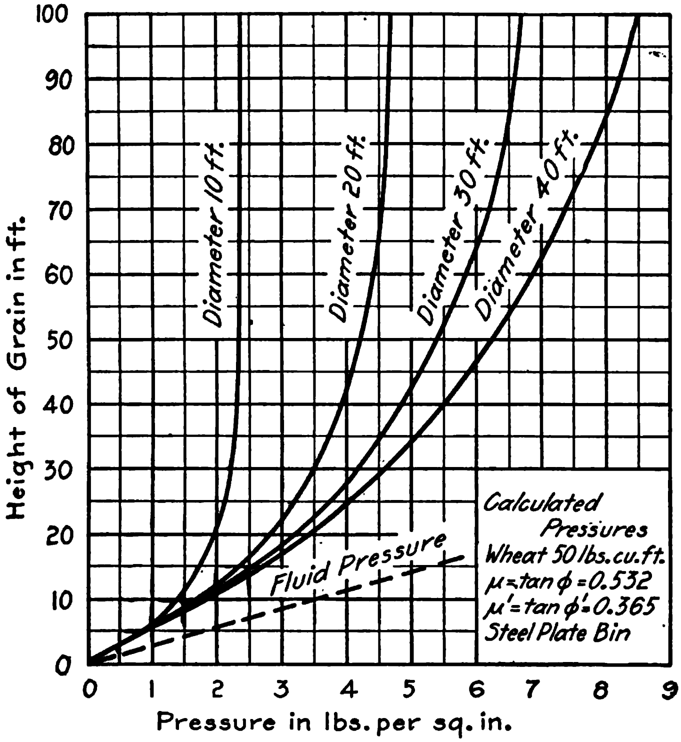


FIG. 154. LATERAL PRESSURE IN STEEL PLATE GRAIN BINS CALCULATED BY JANSSEN'S FORMULA.

TABLE XXXIV.

HYPERBOLIC OR NAPERIAN LOGARITHMS.

N.	Log.	N.	Log.	N.	Log.
1.00	0.0000	3.65	1.2947	6.60	1.8871
1.05	0.0488	3.70	1.3083	6.70	1.9021
1.10	0.0953	3.75	1.3218	6.80	1.9169
1.15	0.1398	3.80	1.3350	6.90	1.9315
1.20	0.1823	3.85	1.3481	7.00	1.9459
1.25	0.2231	3.90	1.3610	7.20	1.9741
1.30	0.2624	3.95	1.3737	7.40	2.0015
1.35	0.3001	4.00	1.3863	7.60	2.0281
1.40	0.3365	4.05	1.3987	7.80	2.0541
1.45	0.3716	4.10	1.4110	8.00	2.0794
1.50	0.4055	4.15	1.4231	8.25	2.1102
1.55	0.4383	4.20	1.4351	8.50	2.1401
1.60	0.4700	4.25	1.4469	8.75	2.1691
1.65	0.5008	4.30	1.4586	9.00	2.1972
1.70	0.5306	4.35	1.4701	9.25	2.2246
1.75	0.5596	4.40	1.4816	9.50	2.2513
1.80	0.5878	4.45	1.4929	9.75	2.2773
1.85	0.6152	4.50	1.5041	10.00	2.3026
1.90	0.6419	4.55	1.5151	11.00	2.3979
1.95	0.6678	4.60	1.5261	12.00	2.4849
2.00	0.6931	4.65	1.5369	13.00	2.5649
2.05	0.7178	4.70	1.5476	14.00	2.6391
2.10	0.7419	4.75	1.5581	15.00	2.7081
2.15	0.7655	4.80	1.5686	16.00	2.7726
2.20	0.7885	4.85	1.5790	17.00	2.8332
2.25	0.8109	4.90	1.5892	18.00	2.8904
2.30	0.8329	4.95	1.5994	19.00	2.9444
2.35	0.8544	5.00	1.6094	20.00	2.9957
2.40	0.8755	5.05	1.6194	21.00	3.0445
2.45	0.8961	5.10	1.6292	22.00	3.0910
2.50	0.9163	5.15	1.6390	23.00	3.1355
2.55	0.9361	5.20	1.6487	24.00	3.1781
2.60	0.9555	5.25	1.6582	25.00	3.2189
2.65	0.9746	5.30	1.6677	26.00	3.2581
2.70	0.9933	5.35	1.6771	27.00	3.2958
2.75	1.0116	5.40	1.6864	28.00	3.3322
2.80	1.0296	5.45	1.6956	29.00	3.3673
2.85	1.0473	5.50	1.7047	30.00	3.4012
2.90	1.0647	5.55	1.7138	31.00	3.4340
2.95	1.0818	5.60	1.7228	32.00	3.4657
3.00	1.0986	5.65	1.7317	33.00	3.4965
3.05	1.1154	5.70	1.7405	34.00	3.5264
3.10	1.1314	5.75	1.7492	35.00	3.5553
3.15	1.1474	5.80	1.7579	40.00	3.6889
3.20	1.1632	5.85	1.7664	45.00	3.8066
3.25	1.1787	5.90	1.7750	50.00	3.9120
3.30	1.1939	5.95	1.7834	60.00	4.0943
3.35	1.2090	6.00	1.7918	70.00	4.2485
3.40	1.2238	6.10	1.8083	80.00	4.3820
3.45	1.2384	6.20	1.8245	90.00	4.4998
3.50	1.2528	6.30	1.8405	100.00	4.6052
3.55	1.2669	6.40	1.8563		
3.60	1.2809	6.50	1.8718		

The author has calculated the lateral pressures on wooden and steel plate bins having diameters or sides of 10, 20, 30, and 40 feet. The lateral pressures on round and square wooden bins are given in Fig. 153, and on round and square steel plate bins in Fig. 154.

To use Fig. 153 and Fig. 154 to calculate the pressures in rectangular bins, calculate the pressure in a circular or square bin which has the same hydraulic radius, R ($R =$ area of bin $\div$ perimeter of bin), as the rectangular bin.

It will be seen in Fig. 153 that the pressure varies as the diameters, where the height divided by the diameter is a constant. By using this principle the pressure for any other diameter, within the limits of the diagram, may be directly interpolated.

Problem 1. Required the lateral pressure at the bottom of a cement lined bin, 10 feet in diameter and 20 feet high, containing wheat weighing 50 lbs. per cu. ft. Assume $\mu' = 0.416$, and $k = 0.6$; also R will $= 2\frac{1}{2}$ feet, $w = 50$ lbs., $h = 20$ feet, and $k.\mu' = 0.25$.
Now from (145)

$$L = \frac{50 \times 2.5}{0.416}\left(1 - e^{-\frac{0.25 \times 20}{2.5}}\right)$$

$$= 300\,(1 - e^{-2})$$

Now the number whose hyperbolic logarithm is 2.00 is 7.40, and

$$L = 300\left(1 - \frac{1}{7.40}\right),$$

$$= 260 \text{ lbs. per square foot,}$$

$$= 1.8 \text{ lbs. per square inch.}$$

German Practice.—Janssen's formula is given in Hütte Des Ingenieurs Taschenbuch, as the standard formula for the design of grain bins. For wheat Janssen found that $\mu' = 0.3$, and $k = 0.67$, so that $\mu'.k = 0.20$. Using these values and changing to English units, we have for wheat,

$$V = \frac{w.R}{0.2}\left(1 - e^{-0.2\frac{h}{R}}\right)$$

or if d = the diameter or side of bin, then

$$V = \frac{5}{4}\,w.d\left(1 - e^{-0.8\frac{h}{d}}\right) \qquad (146)$$

$$L = k.V \qquad (146a)$$

which is the German practice.

AIRY'S SOLUTION.—In a paper presented before the Institute of Civil Engineers, and published in Proceedings, Vol. CXXXI, 1897, Wilfred Airy proposed a method for calculating the pressure of grain on bins. The grain is treated as a semi-fluid according to Weisbach, who assumes the pressure on the wall as due to a wedge of grain between the wall and the plane of rupture. There is friction of grain on the bin wall, and on the plane of rupture. There are two cases depending upon the ratio of width or diameter of the bin to depth of grain: Case I, Shallow Bins, where the plane of rupture cuts the surface of the grain within the bin; and Case II, Deep Bins, where the plane of rupture intersects the side of the bin wall.

Case I. Shallow Bins.—To find the pressure on the sides and bottom of a bin, when the depth of grain in the bin is such that the plane of rupture passes out of the grain before it meets the opposite side of the bin. Let $CABD$ be a vertical section of a bin, Fig. 155, and CD the surface of the grain. Let AE be the plane of rupture. Then ACE is the wedge-shaped mass of grain which causes maximum pressure

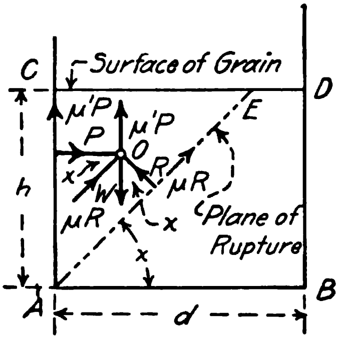


FIG. 155.

against the side *AC*, and *O* its center of gravity. Let *W* be the weight of *ACE*, 1 foot thick, *P* the pressure against the side *AC*, $\mu'.P$ the friction between the grain and the side *AC*, acting in the direction *AC*, μ' being the coefficient of friction, *R* the pressure of the mass *ACE* on the plane of separation *AE*, $V.\mu.R$ the friction between the grain along the plane of separation *AE*, μ being the coefficient of friction, *h* the depth of grain *AC* in feet, *x* the angle *EAB* which the plane of rupture makes with the horizontal, and *w* the weight of one cubic foot of the grain in pounds.

For convenience, the forces are all drawn as if acting at *O* in their proper directions; and resolving the forces that support the mass *ACE* parallel and perpendicular to *AE*,

$$\mu.R + P.\cos x = (W - \mu'.P)\sin x$$

$$R - P.\sin x = (W - \mu'.P)\cos x$$

whence

$$\mu = \frac{(W - \mu'.P)\sin x - P.\cos x}{(W - \mu'.P)\cos x + P.\sin x}$$

and from this by reduction,

$$P = W\,\frac{\tan x - \mu}{1 - \mu.\mu' + (\mu + \mu')\tan x}$$

But

$$W = w \times \tfrac{1}{2}AC \times CE = w \times \tfrac{1}{2}h \times \frac{h}{\tan x} = w \times \frac{h^2}{2\tan x}$$

$$P = w \times \frac{h^2}{2\tan x} \times \frac{\tan x - \mu}{1 - \mu'.\mu + (\mu + \mu')\tan x}$$

$$= w \times \frac{h^2}{2}\,\frac{\tan x - \mu}{(1 - \mu.\mu')\tan x + (\mu + \mu')\tan^2 x}$$

To find the value of *x* that causes *P* to be a maximum, place

$$\frac{dP}{dx} = 0$$

Now

$$\frac{dP}{dx} = w \times \frac{h^2}{2}\left\{\frac{1}{\cos^2 x}\left[(1 - \mu.\mu')\tan x + (\mu + \mu')\tan^2 x\right] - (\tan x - \mu)\right.$$

$$\times \left[\frac{1}{\cos^2 x}(1-\mu.\mu') + 2\tan x \times \frac{1}{\cos^2 x} \times (\mu+\mu')\right]\Bigg\}$$

all divided by

$$\{(1-\mu.\mu')\tan x + (\mu+\mu')\tan^2 x\},$$

therefore

$$(1-\mu.\mu')\tan x + (\mu+\mu')\tan^2 x - (1-\mu.\mu')(\tan x - \mu)$$

$$-2\tan x\,(\tan x - \mu)(\mu+\mu') = 0;$$

whence by reduction,

$$\tan x = \mu + \sqrt{\mu \times \frac{1+\mu^2}{\mu+\mu'}},$$

since the part under the radical is greater than μ. If this value of $\tan x$ is substituted in the expression for P, the lateral pressure of the wedge ACE, 1 foot thick, will be obtained; and this result multiplied by the horizontal circumference of the bin in feet, gives the total pressure on the sides of the bin. This, multiplied by μ', gives the vertical sustaining force of the side friction; and the pressure on the bottom is the total weight of the grain less the vertical sustaining force of the friction. Thus the fundamental equations for Case I are:

$$\tan x = \mu + \sqrt{\mu \times \frac{1+\mu^2}{\mu+\mu'}} \qquad (147)$$

$$P = w \times \frac{h^2}{2\tan x} \times \frac{\tan x - \mu}{1-\mu.\mu' + (\mu+\mu')\tan x} \qquad (148)$$

Case II. Deep Bin.—To find the pressure on the sides and bottom of a bin, when the depth of grain is such that the plane of rupture meets the oposite side of the bin within the mass of grain. Let d be the breadth of the bin, Fig. 156. Then, as in Case I,

$$P = W \times \frac{\tan x - \mu}{1-\mu.\mu' + (\mu+\mu')\tan x}$$

But

$$W = w \times 1 \times \text{area } AEDCA = w \times CD \times \frac{AC+DE}{2}$$

$$= w \times \frac{d}{2}\,(2h - d\,.\tan x)$$

$$P = w \times \frac{d}{2}\,(2h - d\,.\tan x) \times \frac{\tan x - \mu}{1 - \mu.\mu' + (\mu + \mu')\tan x}$$

or

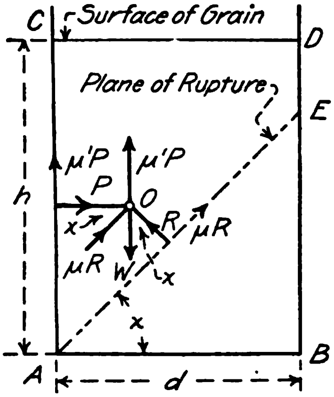


FIG. 156.

$$P = w \times \frac{d}{2}\left\{2h \times \frac{\tan x - \mu}{1 - \mu.\mu' + (\mu + \mu')\tan x} - d \times \frac{\tan^2 x - \mu\,.\tan x}{1 - \mu.\mu' + (\mu + \mu')\tan x}\right\}$$

To find the value of x which makes P a maximum, place $\frac{dP}{dx} = 0$. Now.

$$\frac{dP}{dx} = w \times \frac{d}{2}\left\{2h\left[\frac{1}{\cos^2 x}(1 - \mu.\mu') + \frac{1}{\cos^2 x}(\mu + \mu')\tan x - \frac{1}{\cos^2 x}(\mu + \mu')\tan x + \frac{1}{\cos^2 x}(\mu + \mu')\mu\right]\right.$$

$$\left. - d\left[\left(2\tan x \times \frac{1}{\cos^2 x} - \mu \times \frac{1}{\cos^2 x}\right)\left\{1 - \mu.\mu' + (\mu + \mu')\tan x\right\} - \frac{1}{\cos^2 x}(\mu + \mu')(\tan^2 x - \mu\,.\tan x)\right]\right\}$$

all divided by

$$\{1-\mu.\mu'+(\mu+\mu')\tan x\}^2$$

so that

$$2h\,(1+\mu^2)-d\{\tan^2 x(\mu+\mu')+2\tan x(1-\mu.\mu')-\mu(1-\mu.\mu')\}=0,$$

which by reduction gives finally

$$\tan x=\sqrt{\frac{2h}{d}\times\frac{1+\mu^2}{\mu+\mu'}+\frac{1+\mu^2}{\mu+\mu'}\times\frac{1-\mu.\mu'}{\mu+\mu'}}-\frac{1-\mu.\mu'}{\mu+\mu'}$$

Substituting this value of tan x in the expression for P, the maximum pressure on the side of the bin of the wedge-shape mass $AEDCA$, 1 foot thick, is obtained; and the pressure on the sides and bottom of the bin can be deduced as before. Thus the fundamental equations for Case II are:

$$\tan x=\sqrt{\frac{2h}{d}\times\frac{1+\mu^2}{\mu+\mu'}+\frac{1+\mu^2}{\mu+\mu'}\times\frac{1-\mu.\mu'}{\mu+\mu'}}-\frac{1-\mu.\mu'}{\mu+\mu'} \qquad (149)$$

$$P=w\times\frac{d}{2}\times(2h-d.\tan x)\times\frac{\tan x-\mu}{1-\mu.\mu'+(\mu+\mu')\tan x} \qquad (150)$$

It is seen from Fig. 156 that the value of h for which the plane of separation of the mass of maximum pressure meets the opposite side at the surface level of the grain, is determined by the condition $h=d.\tan x$. If in the expression for $\tan x$ in Case II, $d.\tan x$ is substituted for h, then

$$\tan x=\sqrt{2\tan x\times\frac{1+\mu^2}{\mu+\mu'}+\frac{1+\mu^2}{\mu+\mu'}\times\frac{1-\mu.\mu'}{\mu+\mu'}}-\frac{1-\mu.\mu'}{\mu+\mu'}$$

or,

$$\tan x=\mu+\sqrt{\mu\,\frac{1+\mu^2}{\mu+\mu'}} \qquad (147)$$

the same expression as in Case I. Similarly the value of P from Case II becomes, by putting $h=d.\tan x$ or $d=\frac{h}{\tan x}$

$$P=w\times\frac{h^2}{2\tan x}\times\frac{\tan x-\mu}{1-\mu.\mu'+(\mu+\mu')\tan x} \qquad (148)$$

the same expression again as was obtained in Case I, showing that the equations for $\tan x$ and P in Case I and Case II are continuous.

A useful law connecting the pressures in bins of different sizes can be obtained immediately from the equations as follows: In Case I, taking the case of a square bin, the length of side of which is d, the total pressure on the sides of the bin is $P \times 4d$. Calling this n, the second equation, Case I, becomes

$$n = 2w \times h^2.d \times \frac{\tan x - \mu}{(1 - \mu.\mu') \tan x + (\mu + \mu') \tan^2 x}$$

which may be written

$$n = 2w \times \frac{h^2}{d^2} \times d^3 \times \frac{\tan x - \mu}{(1 - \mu.\mu') \tan x + (\mu + \mu') \tan^2 x} \quad (151)$$

therefore, in comparing the pressures on the sides of square bins of different sizes, if $\frac{h}{d}$ (the proportion of the depth of the grain to the side of the bin) is the same for both, the pressure varies as d^3, or the pressure per unit of length as d^2.

In Case II, with square bins as before, calling $n = P \times 4d$, the total pressure on the sides of the bin is

$$n = 2w \times d^3 \left(\frac{2h}{d} - \tan x\right) \frac{\tan x - \mu}{1 - \mu.\mu' + (\mu + \mu') \tan x} \quad (152)$$

Therefore, in this case also, in comparing the total pressures on the sides of square bins of different sizes, if $\frac{h}{d}$ is the same for both, the pressure varies as d^3. The total weight of grain in the bin is $w.h.d^2$, and the pressure on the bottom of the bin is $w.h.d^2 - \mu'.n$. Then comparing square bins, for which $\frac{h}{d}$ is constant, say $\frac{h}{d} = C$, the pressure on the bottom $= w \times d^3.C - m.d^3 = d^3(w.C - m)$, m being a constant; therefore the pressure on the bottom also varies as d^3, or the pressure per unit of length varies as d^2.

The pressure per lineal unit of circumference of a circular bin, having the diameter d, will be the same as the pressure per lineal unit

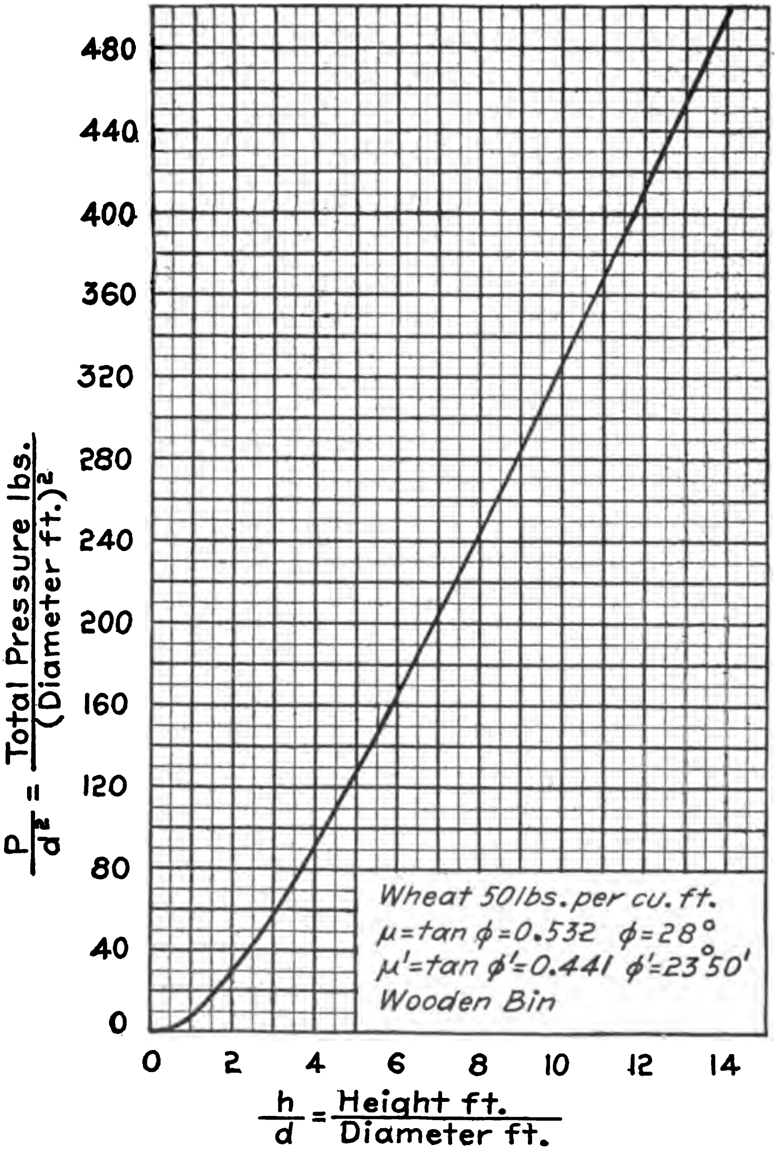


FIG. 157. DIAGRAM FOR CALCULATING THE LATERAL PRESSURE ON BIN WALLS ONE FOOT LONG, BY AIRY'S FORMULAS.

of a square bin. Therefore in circular bins in which $\frac{h}{d}$ is a constant, the total pressure on the walls of the bin varies as d^3, and the pressure per unit of length of circumference varies as d^2.

Graphic Solution.—The formulas deduced above are too complicated for practical use. In Fig. 157 the author has tabulated the pressures for wheat in a wooden bin, using h/d as abscissas and P/d^2 as ordinates, where P is the total pressure on a bin wall one foot long and of the depth h. To calculate the pressure on a square foot, at a depth of say 50 feet, calculate P for $h = 50$ feet and for $h = 49$ feet,

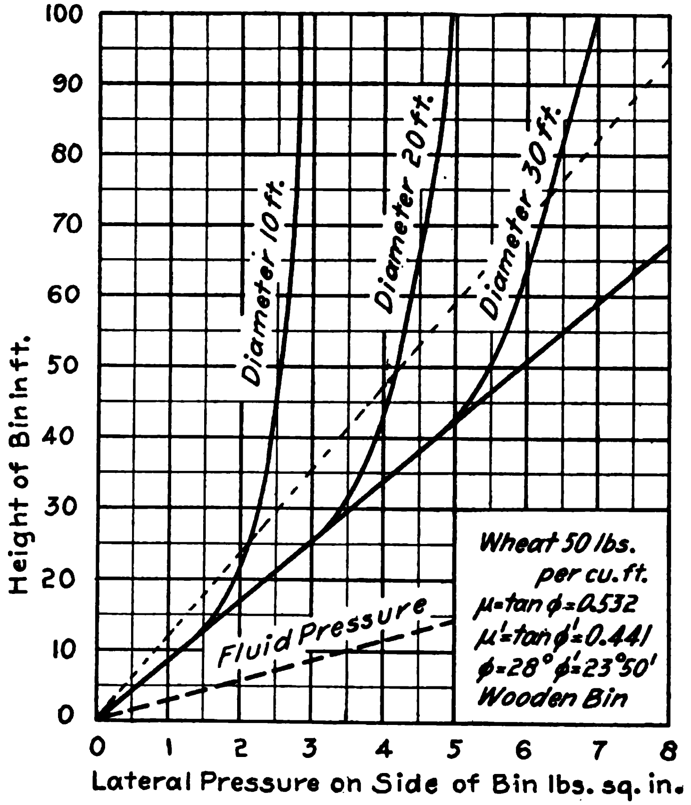


FIG. 158. LATERAL PRESSURE OF WHEAT ON WALLS OF WOODEN BINS, CALCULATED BY AIRY'S FORMULAS.

and the difference in pressure will be the pressure on the square foot at the required depth. In this manner the author has calculated, in Fig. 158, the lateral pressure per square inch for wheat on wooden bin walls, for bins having a diameter, or side of square, of 10, 20, and 30 feet. By comparing Fig. 158 with Fig. 153, it will be seen that the pressures as calculated by Airy's formula are somewhat larger than when calculated by Janssen's formulas. The diagram in Fig. 157 may be used for calculating the pressures in concrete and tile bins without change, and will give results for steel plate bins if the results are multiplied by $\frac{4}{5}$. Fig. 157 and Fig. 158 may be used for calculating the pressure of corn in bins without change.

It will be seen in Fig. 158 that the pressure varies as the diameter when the height divided by the diameter, $\frac{h}{d}$, is a constant. By using this principle the pressures on bins of other diameters may be interpolated directly from the table. For example, the maximum lateral pressure on a bin 15 feet in diameter and 45 feet deep is 3.3 lbs. per square inch.

Airy's solution applies to rectangular bins, the unit pressures on a side being the same as on a square bin having dimensions equal to the other side of the rectangular bin.

Vertical Pressures.—The vertical pressures in a deep grain bin are calculated as follows: The grain supported by the side walls is equal to the total lateral pressure multiplied by μ', the coefficient of friction of the grain on the bin wall. The grain carried on the bottom of the bin is equal to the total weight of grain, minus the grain carried by the side walls. This bottom pressure is not uniformly distributed, but is a minimum at the side walls, and a maximum at the center of the bin. The grain mass producing bottom pressure might be represented by a portion of an ellipsoid of revolution, with the major axis of the ellipse vertical. The truth of this is shown by Bovey's Experiments, Chapter XVII.

CHAPTER XVII.

EXPERIMENTS ON THE PRESSURE OF GRAIN IN DEEP BINS.

Introduction.—The law of pressure of grain and similar materials is very different from the well known laws of fluid pressure. Dry wheat and corn come very nearly filling the definition of a granular mass assumed by Rankine in deducing his formulas for earth pressures. As stored in a bin the grain mass is limited by the bin walls, and Rankine's retaining wall formulas are not directly applicable.

If grain is allowed to run from a spout onto a floor it will heap up until the slope reaches a certain angle, called the angle of repose of the grain, when the grain will slide down the surface of the cone. If a hole be cut in the bottom of the side of a bin, the grain will flow out until the opening is blocked up by the outflowing grain. There is no tendency for the grain to spout up as in the case of fluids. If grain be allowed to flow from an orifice it flows at a constant rate, which is independent of the head and varies as the diameter of the orifice.

Experiments by Willis Whited,* and of the author at the University of Illinois, with wheat have shown that the flow from an orifice is independent of the head and varies as the cube of the diameter of the orifice. This phenomenon can be explained as follows: The wheat grains in the bin tend to form a dome which supports the weight above. The surface of this dome is actually the surface of rupture. When the orifice is opened the grain flows out of the space below the dome and the space is filled up by grains dropping from the top of the dome. As these grains drop others take their place in the dome. Experiments with glass bins show that the grain from the center of the bin is discharged first, this drops through the top of the dome, while the grain in the lower part of the dome discharges last.

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* *Proc. Eng. Soc. of West Penna.*, April, 1901.

The law of grain pressures has been studied experimentally by several engineers within recent years. A brief résumé of the most important experiments follows:

ROBERTS' EXPERIMENTS.*—The first recorded experiments on the pressure of wheat in bins were made by Isaac Roberts in 1882 and 1884. His first experiments were made on small bins, 7 to 20¾ inches in diameter. He observed that "all increase of pressure on the bottom ceases before the cells are filled two diameters." These experiments caused him to make a second series on a rectangular bin 6′ 9″ × 6′ 0″ and 52′ 2″ high. The pressures measured were the pressures on square orifices: 1′ 0″ × 1′ 0″, 2′ 0″ × 2′ 0″, and 3′ 0″ × 3′ 0″. For the bottom pressures the orifices were central, and for the side pressures the orifices were at the bottom of the side. The pressures were measured by levers arranged like a weighing machine. He found considerable difficulty in measuring the pressures with this apparatus, apparently due to the elasticity of the wheat and the movement necessary to raise the lever. To measure the pressure excess weights were placed on the scale, then the grain was put in the bin, and the scale weights were removed one at a time until the scale beam arose. It was found that the beam started at a certain pressure but in order to raise the scale beam it was necessary to remove one weight after another until the pressure indicated was very much less than the initial pressure which started the scale beam. This difficulty is explained in detail because it has in other cases caused erroneous results, as for example, in Prante's experiments.

The second set of experiments by Roberts appeared to confirm the conclusion above—that there is no increase in bottom pressure after the grain has a depth more than twice the width of the bin.

JANSSEN'S EXPERIMENTS.†—H. A. Janssen of Bremen made experiments in 1895 to determine the pressure of grain on bin walls.

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* *Engineering*, October 27, 1882, and *Proceedings Royal Society of London*, January 31, 1884.

† "Versuche über Getreidedruck in Silozellen," *Zeitschrift des Vereines deutscher Ingenieure*, 1895, p. 1045.

His bins were square model bins having sides 20, 30, 40, and 60 centimeters in length, respectively. The bin walls were supported on 4 jack screws and the loose bottoms were supported directly on a scale. By filling the bin with grain the proportion of the weight resting on the bottom was recorded on the scale. The bin was then slightly raised by the jack screws, and, owing to the friction of the grain on the sides of the bin, this also relieved part of the bottom pressure and allowed the beam to drop. Added weights were then placed on the beam and the filling of the bin proceeded. Janssen made experiments on wheat, corn, and other grains; also on dry sand. The bottom pressures were measured as above and the horizontal pressures were calculated by means of Janssen's formulas (144) and (145). The pressures of wheat on a bin 30 cm. × 30 cm. (11.8 × 11.8 inches) as determined by Janssen are given in Fig. 159.

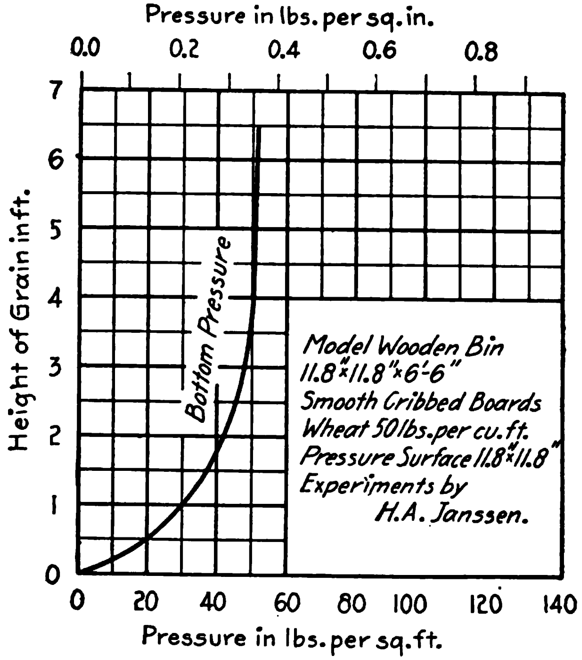

FIG. 159. EXPERIMENTS ON THE PRESSURE OF WHEAT IN A MODEL BIN.

Janssen developed the formulas given in Chapter XVI. The ratio of lateral to vertical pressures for dry wheat as determined from his experiments was $k = 0.67$, and the coefficient of friction of the wheat on the wooden bin walls was $\mu' = 0.3$. The value of k as determined by these experiments appears to be too large for full-sized bins.

Janssen's formulas (144) and (145) and his coefficients are used in grain bin design in Germany.

PRANTE'S EXPERIMENTS.*—Prante's experiments were made on two full-sized cylindrical iron bins, 1.5 and 3.8 meters in diameter and 19 meters high. The pressure measurement apparatus consisted of a diaphragm supported on knife edges and connected by a system of levers to a scale pan. Side pressures only were measured. The apparatus as first constructed was not satisfactory, and the details were later slightly changed. These experiments gave the lateral pressure of wheat at rest slightly smaller than the values calculated by Janssen's formulas (144) and (145). With wheat in motion with a velocity in the bin of 1 mm. per second, however, the side pressures rapidly increased until they were as much as four times the static pressures. The discharge gate was located near the bottom of the side, probably just opposite the pressure measurement opening. This would result in pressures on the sides opposite the gate very much in excess of the static pressure. While Prante's experiments were quite elaborate the results may not be reliable on account of the measuring apparatus and the location of the discharge gate. The experiments show that bins should be constructed with discharge gates at or near the center of the bottom of the bin.

Prante's experiments are not considered as reliable in Germany where Janssen's formulas and coefficients are used in the design of grain bins.

TOLZ'S EXPERIMENTS.†—In designing the Great Northern Elevator, Buffalo, N. Y., in 1897, Max Tolz, mechanical engineer Great Northern Ry., based the design on Prante's experiments, and designed the bins for fluid pressure. Before building the Great Northern elevator, West Superior, Wis., Mr. Tolz made a series of experiments on the pressure of wheat in a wooden bin 14′ 0″ square and 65′ 0″ deep. In one of the walls near the bottom a hole was cut 1′ 6″ × 3′ 0″, and in the opening a steel plate was placed, which was held rigidly at two ends by steel channels, but the top and bottom edges were free. The bin was then gradually filled with grain, the deflections of the plate for different heights of grain being carefully measured. The process of filling and emptying the bin was repeated several times, using plates Nos. 12, 15, 18, and 22 gage. By testing the plates it was found that the lateral pressures, either when filling or emptying the bin did not exceed 3 pounds per square inch. These experiments were very carefully conducted and the results may be taken as reliable.

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* "Messungen des Getreidedruckes in Silozellen," *Zeitschrift des Vereines deutscher Ingenieure*, 1896, p. 1192.

† *Trans. Can. Soc. C. E.*, Vol. XVII, 1903.

AIRY'S EXPERIMENTS.—In a paper to Institute of Civil Engineers, and printed in Proceedings, Vol. CXXXI, 1897, Wilfred Airy gives a valuable discussion on the theory of grain pressures and also the results of a series of experiments to determine the angle of repose of grains, and the coefficient of friction of grain on bin walls. The angle of repose of grain, ϕ, was determined by measuring the slopes of grain in piles. The coefficient of friction, $\mu' = \tan \phi'$, was determined by finding the slope at which a piece of the bin wall would slide down the grain slope. The results of these experiments are given in Table XXXV.

TABLE XXXV.

COEFFICIENTS OF FRICTION OF VARIOUS KINDS OF GRAIN ON BIN WALLS.

	Weight of a Cubic Foot Loosely Filled Into Measure. Pounds.	Coefficients of Friction.				
		Grain on Grain. μ. $\tan \phi$.	Grain on Rough Board μ'. $\tan \phi'$.	Grain on Smooth Board μ'. $\tan \phi'$.	Grain on Iron μ'. $\tan \phi'$.	Grain on Cement μ'. $\tan \phi'$.
Wheat..........	49	0.466	0.412	0.361	0.414	0.444
Barley	39	0.507	0.424	0.325	0.376	0.452
Oats	28	0.532	0.450	0.369	0.412	0.466
Corn............	44	0 521	0.344	0.308	0.374	0.423
Beans...........	46	0.616	0.435	0.322	0.366	0.442
Peas	50	0.472	0.287	0.268	0.263	0.296
Tares..........	49	0.554	0.424	0.359	0.364	0.394
Flaxseed	41	0.456	0.407	0.308	0.339	0.414

THE AUTHOR'S EXPERIMENTS.—In 1902 and 1903 a series of experiments on grain pressure was carried on at the University of Illinois by Albert G. Varnes, under the author's direction. The experiments were made with a model bin 1′ 0″ × 1′ 0″ and 8′ 6″ high, made of dressed white pine with the grain vertical. The bottom was movable and the bottom pressures were measured by supporting the movable bottom directly on a platform scale. The side pressures were measured on a diaphragm 12 inches square placed in one side near the bottom. The pressure was transmitted to a platform scale by means of levers.

Experiments were first made on the flow of wheat from an orifice, and it was found that the flow was independent of the head and varied as the cube of the orifice.

Experiments were then made to determine the side pressures and bottom pressures, when the grain was at rest, and in motion. To obtain the pressure of moving grain the bin was filled and the scale beam balanced. Then the orifice was opened and the wheat was allowed to flow out, the drop of the beam being noted at intervals together with the time that had elapsed since the grain started to flow. The orifice having been calibrated, the height of the grain corresponding to the scale readings was easily determined. Bottom pressures were determined with both side and bottom orifices, while the side pressures were determined with a bottom orifice only; the orifices being 2″ in diameter. The pressures for the bin filling were determined by placing a hopper above the bin, from which the grain ran into the bin through an orifice.

It was not possible to detect any difference in pressure for the grain at rest or in motion. The means of four tests are given in Fig. 160. It will be seen that the ratio of lateral to vertical pressure is very nearly $k = 0.4$.

JAMIESON'S EXPERIMENTS.*—In 1900 J. A. Jamieson, Montreal, Canada, made a series of experiments on a full-sized bin of the Canadian Pacific Ry. Elevator, West St. John, N. B. The bin was of

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* *Trans. Can. Soc. C. E.*, Vol. XVII, 1903; *Eng. News*, 1904, Vol. 51, p. 236.

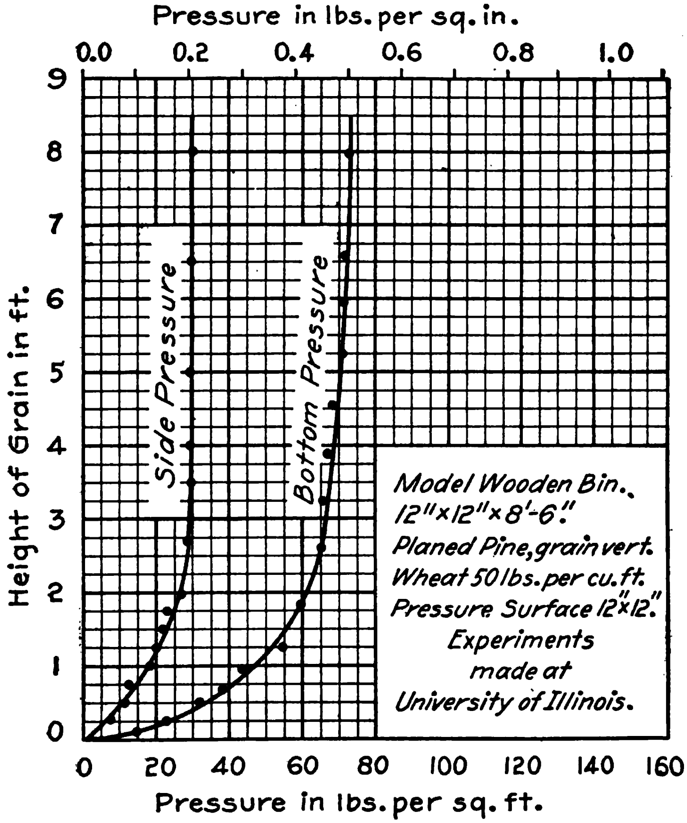

FIG. 160. EXPERIMENTS ON THE PRESSURE OF WHEAT IN A MODEL BIN MADE AT THE UNIVERSITY OF ILLINOIS, GRAIN IN MOTION.

timber crib construction, 12′ 0″ × 13′ 6″ in cross section and 67′ 6″ high. The grain used was Manitoba wheat weighing 49.4 pounds per cubic foot.

To obtain the pressure of the grain, diaphragms as shown in Fig. 161 were used. To obtain the lateral pressure the diaphragm was placed in position a short distance above the hopper bottom, as shown in Fig. 161. To obtain the bottom pressure the diaphragm was placed horizontal on top of a small platform attached to the hopper bottom. The grain was run into the bin in drafts of 30,000 pounds, giving a depth of 3′ 9″ in the bin. The gage was observed after each draft and the maximum reading recorded. After the bin was filled it was allowed to stand 18 hours, during which time the gage reading was

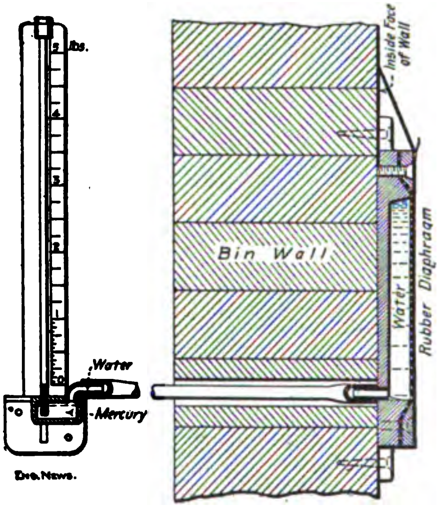

FIG. 161. HYDRAULIC PRESSURE DIAPHRAGM AND MERCURY GAGE.

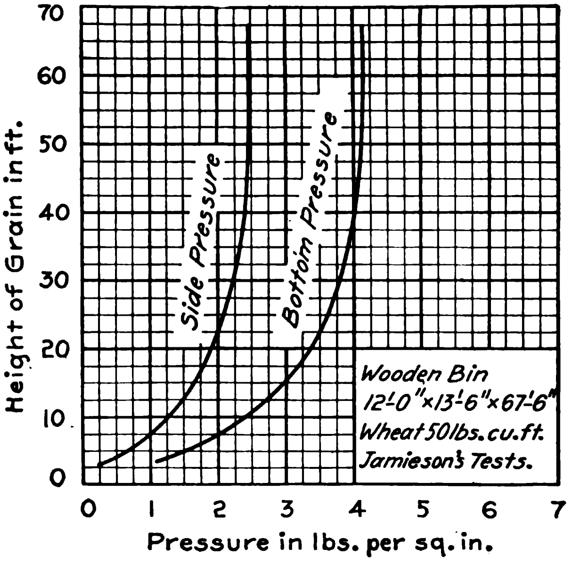

FIG. 162. JAMIESON'S TESTS ON CANADIAN PACIFIC RY. GRAIN BIN.

constant. The grain was then drawn off through the discharge gate in the center of the bottom in 30,000-lb. lots, at the rate of 9,000 bushels per hour. The pressures during emptying fluctuated considerably, the maximum increase of pressure in emptying over the pressure in filling was 4 per cent. Closing the valve during emptying gave a slight increase in pressure, opening the valve a slight decrease. The average pressures of the wheat on the side and bottom of the bin are given in Fig. 162. The coefficient of friction between the grain and the sides was $\mu' = 0.441$. The angle of repose of the grain was $\phi = 28°$, and $\mu = 0.532$. The ratio of the lateral to the vertical pressure was $k = 0.596$, or the vertical pressure = 1.66 times the lateral pressure.

The calculated pressures, using Janssen's formulas (144) and (145), using $k = 0.6$, on the same grain bin are given in Fig. 163. The calculated pressures agree very closely with the observed pressures.

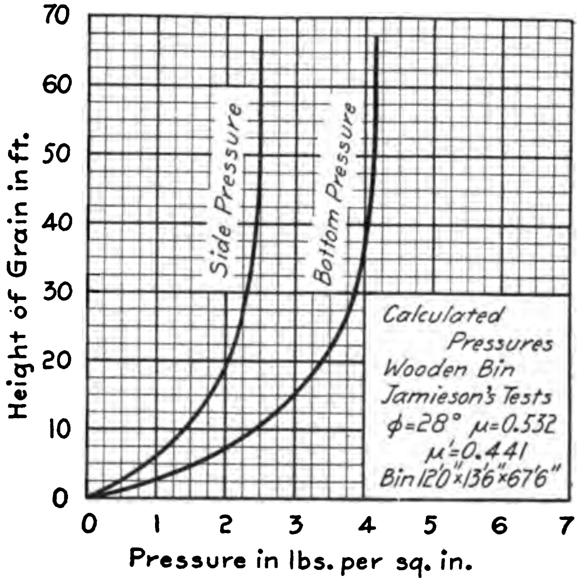

FIG. 163. CALCULATED PRESSURES OF WHEAT, USING JANSSEN'S FORMULAS, ON CANADIAN PACIFIC RY. GRAIN BIN.

Model Bins.—Jamieson also made experiments on model bins. Experiments were made with bins constructed of smooth boards cribbed,

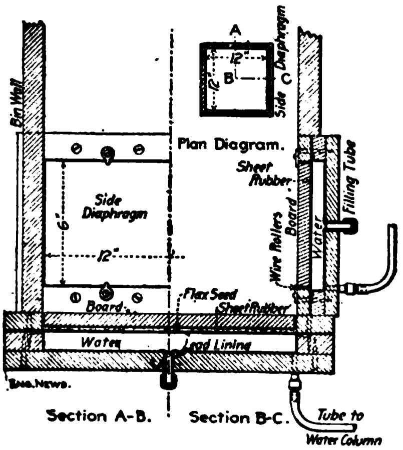

FIG. 164. MODEL BIN AND DIAPHRAGM FOR DETERMINING GRAIN PRESSURE.

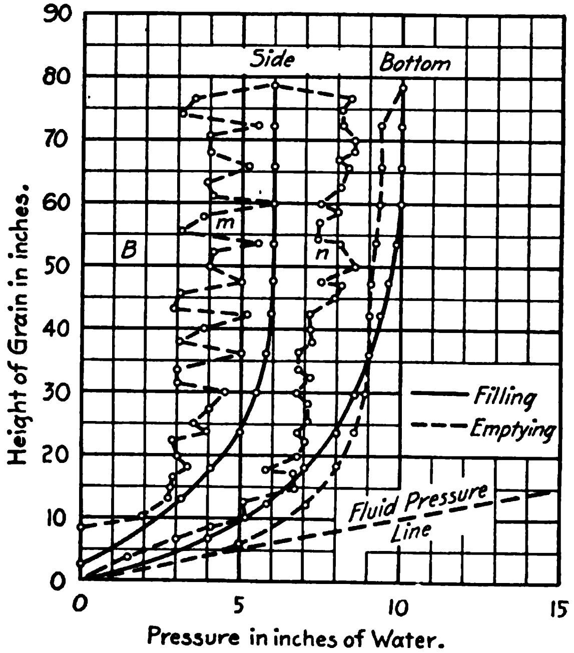

FIG. 165. CURVES SHOWING VARIATIONS IN GRAIN PRESSURES IN FILLING AND EMPTYING BINS WITH OPENING AT SIDE OF BOTTOM. (CURVE *m* SHOWS PRESSURE ON SIDE ADJACENT TO OUTLET AND *n* ON SIDE OPPOSITE.)

flat steel plates, and trough plate bins, the corrugations running horizontally. The bins were 12″ and 6″ square or round, and 6′ 6″ high. The arrangement of the bins and the diaphragms are shown in Fig. 164 and Fig. 167. In testing the bottom pressure the diaphragm covered the entire bottom of the bin. In testing the lateral pressure the diaphragm was made a part of the bin wall, the face being set vertical and flush with the inside of the bin. Grain was poured in at the top of the bin in drafts of 6¼ to 25 pounds, depending on the size of the bin, and the height of the water column was recorded for each draft. Tests were all made in duplicate, one set being made with the grain in direct contact with the rubber and the other with a board placed over the rubber to distribute the pressure, as shown in Fig. 164. By tapping the bin with a hammer near the bottom the pressure on the bottom could be decreased. When the tapping was continued from

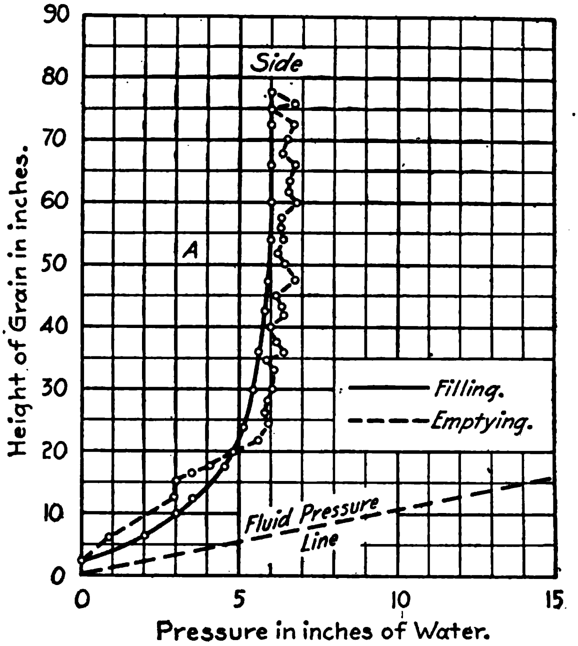

FIG. 166. LATERAL PRESSURE OF WHEAT AT REST AND IN MOTION, OPENING IN CENTER OF BOTTOM.

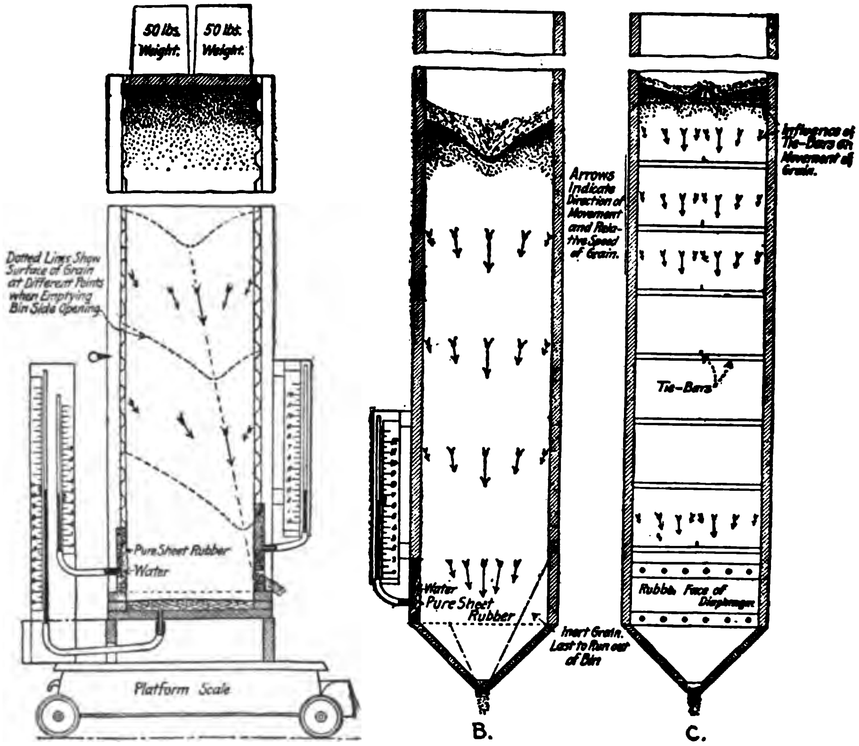
50 lbs. Weight.
50 lbs. Weight.
Dotted Lines Show Surface of Grain at Different Points when Emptying Bin Side Opening.
Pure Sheet Rubber
Water
Platform Scale
Arrows Indicate Direction of Movement and Relative Speed of Grain.
Water
Pure Sheet Rubber
Inert Grain. Last to Run out of Bin
B.
Influence of Tie-Bars on Movement of Grain.
Tie-Bars
Rubber Face of Diaphragm
C.

FIG. 167. Model Bin and Flow of Grain in a Bin.

the bottom to the top on all sides the grain settled 2 to 3 inches, and the bottom pressure was slightly increased.

By placing standard weights of 50 pounds each on top of the grain, the bottom pressure could be only slightly increased by each weight added, the pressure becoming normal on removing the weights. Experiments were made to determine the effect of moving grain on the pressure. The variation of pressure of wheat for grain at rest and in motion, with the outlet in one side of the bottom, is given in Fig. 165, and with the opening in the center of the bottom in Fig. 166. The action of the grain in flowing out of the bin is shown in Fig. 167. It was found that when the grain was drawn from one side of the bin, the pressure was decreased on the side of the opening and increased on the opposite side. When the grain was taken from the middle of the bottom the increase in pressure for grain running at a rate of from 50 to 120 lbs. per minute was from 5 to 7.3 per cent. Jamieson places the maximum increase in pressure due to moving grain in full-sized bins at 10 per cent, where the ratio of area of the gate opening to the area of the bin is not greater than 1 : 150. Tests were also made with

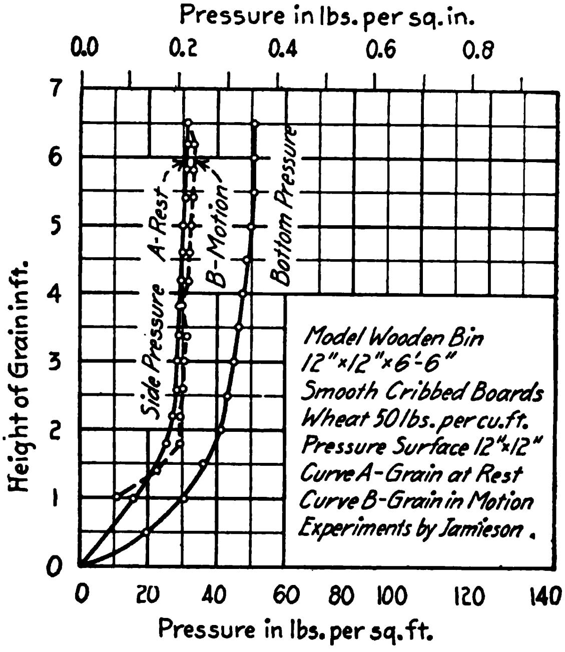

FIG. 168. PRESSURE OF WHEAT IN A MODEL WOODEN BIN.

tie bars of 24 gage steel ½ inch wide, placed on edge across the bins and spaced 6 inches apart. It was found that the tie bars reduced the amount of grain running out of the bottom about one-third, but did not affect the pressures of the grain.

The pressures on a model wooden bin for wheat at rest and in motion are given in Fig. 168. It will be seen that the pressure is only slightly greater when the grain is in motion than when at rest. The percentage of wheat carried by the bottom and sides of the model

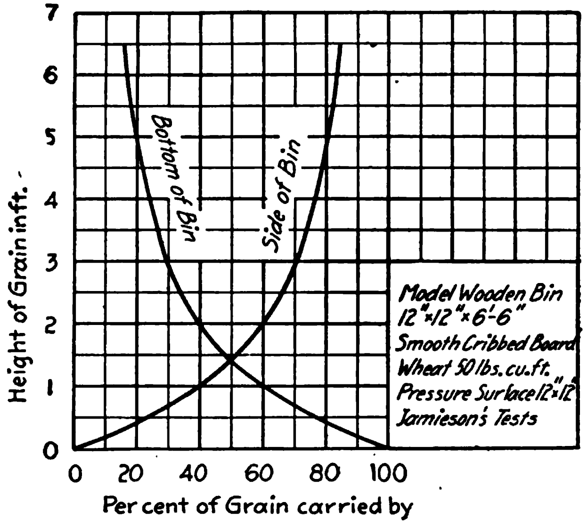

FIG. 169. THE PERCENTAGE OF WHEAT CARRIED BY BOTTOM AND SIDES IN A MODEL WOODEN BIN.

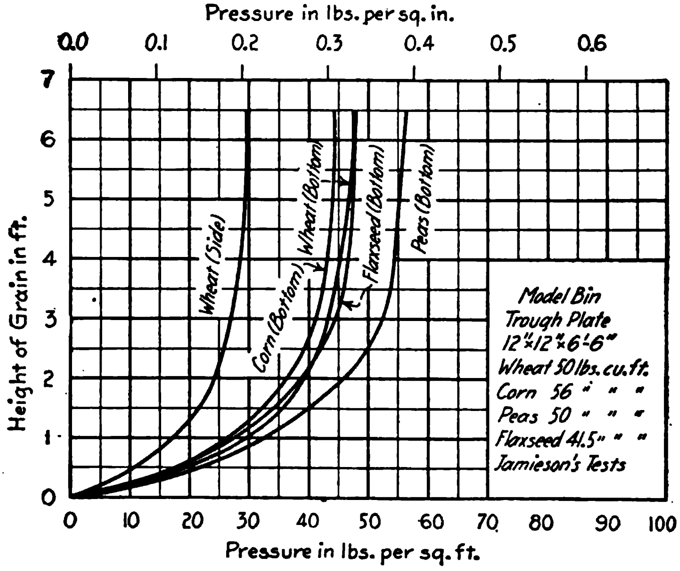

FIG. 170. TESTS OF PRESSURE OF GRAIN IN A MODEL STEEL TROUGH PLATE BIN.

wooden bin are shown in Fig. 169. It will be seen that the amount carried by the bottom and sides is equal at a height of about 1.4 times the side of the bin.

Tests were made on a square trough plate bin, 12 inches square and 6′ 6″ deep, with wheat, peas, corn, and flax-seed. The weights of the grain per cubic foot were: wheat, 50 lbs.; peas, 50 lbs.; corn, 56 lbs.; and flax-seed, 41.5 lbs. The tests are shown in Fig. 170. Jamieson

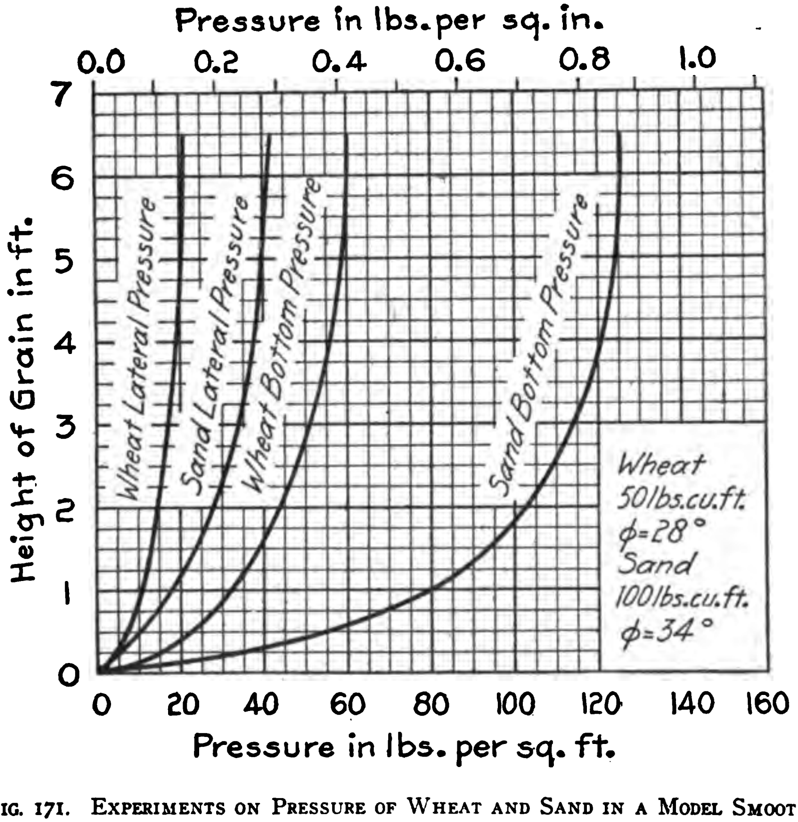

FIG. 171. EXPERIMENTS ON PRESSURE OF WHEAT AND SAND IN A MODEL SMOOTH STEEL BIN 12″ DIAMETER.

states that taking wheat weighing 50 lbs. per cubic foot as a standard, corn weighing 56 lbs. per cubic foot will give approximately the same pressure as wheat; peas weighing 50 lbs. per cubic foot will give approximately 20 per cent greater pressure than wheat, while flax-seed weighing 41.5 lbs. per cubic foot will give from 10 to 12 per cent greater pressure than wheat. The coefficients of friction of the different grains vary, which explains the difference in pressure.

The tests of the pressure of dry sand, weighing 100 lbs. per cubic foot, and wheat, weighing 50 lbs. per cubic foot, on a cylindrical bin of smooth steel 12″ in diameter are shown in Fig. 171.

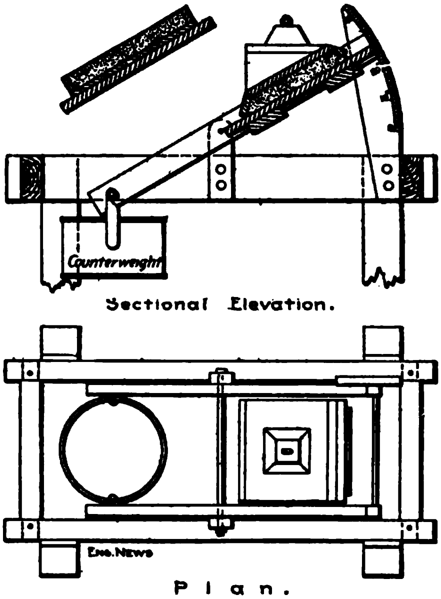

FIG. 172. APPARATUS FOR TESTING THE ANGLE OF REPOSE AND COEFFICIENT OF FRICTION OF GRAINS ON BIN WALLS.

Angle of Repose and Coefficient of Friction.—Jamieson made careful tests with the apparatus shown in Fig. 172 to determine the angle of repose of grains, ϕ, and the coefficient of friction, $\mu' = \tan \phi'$, of grains on bin walls. To obtain the angle of repose of grain the tray, which was attached to the pivoted frame, was filled and carefully leveled off, and the frame balanced; the end holding the tray was then carefully raised until the first movement of the grain took place and this angle noted as the angle of repose. By attaching to the pivoted frame a piece of the bin wall material, and again filling the tray with grain, with the tray inverted and with the grain against the material, by raising the pivoted frame the angle of friction of the grain on the material will be found when the tray begins to move. The means of the tests are given in Table XXXVI.

TABLE XXXVI.

COEFFICIENTS OF FRICTION AND ANGLE OF REPOSE OF WHEAT. WHEAT WEIGHING 50 LBS. PER CUBIC FOOT, AND ANGLE OF REPOSE $\phi = 28$ DEGREES.

Materials.	Coefficient of Friction.
Wheat on wheat	0.532
Wheat on steel trough plate bin	0.468
Wheat on steel flat plate, riveted and tie bars	0.375 to 0.400
Wheat on steel cylinders, riveted	0.365 to 0.375
Wheat on cement-concrete, smooth to rough	0.400 to 0.425
Wheat on tile or brick, smooth to rough	0.400 to 0.425
Wheat on cribbed wooden bin	0.420 to 0.450

BOVEY'S EXPERIMENTS.*—Professor Henry T. Bovey, McGill University, Montreal, made a series of experiments on full-sized bins in 1901, to check up Jamieson's tests. Four series of tests were made on the Canadian Pacific Ry. grain elevator in Montreal, and one series was made on the Great Northern R. R. grain elevator in Quebec.

The pressures were determined with flat rubber diaphragms similar to those used by Jamieson, the apparatus having been carefully calibrated before and after the experiments.

Series I to IV, Canadian Pacific Ry. Elevator, Montreal.—The bin used was 12 × 14 ft. in cross-section with a hopper gate 12 inches in diameter. Two diaphragms, (B) and (C), were used. Diaphragm (B), having 109.36 sq. in. effective sectional area, was placed horizontally in the center of the bin, and the other, (C), having 26 sq. in. effective sectional area, was placed vertically in the side of the bin, in such a position that the maximum height of grain above the centers of both diaphragms was 44 ft. 10 in. The hopper was filled to the proper height and diaphragm (B) was laid on the grain at the center. The pressures were determined with the grain in motion, and the effect of sudden stopping after each 500 bushels was observed. The sudden stopping produced no appreciable effect.

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* *Trans. Can. Soc. C. E.*, Vol. XVII, 1903; *Eng. News*, 1904, Vol. 52.

The lateral and vertical pressures at bottom of bin when filling are given in Fig. 173. The wheat was allowed to stand one day and the bin was emptied. The vertical pressures could not be determined, as diaphragm (*B*) dropped out as soon as the gate was opened. The maximum lateral pressure for bin emptying was 2.142 lbs. per sq. in. as compared with a maximum lateral pressure of 1.951 lbs. per sq. in. when the bin was filling (9.7 per cent increase).

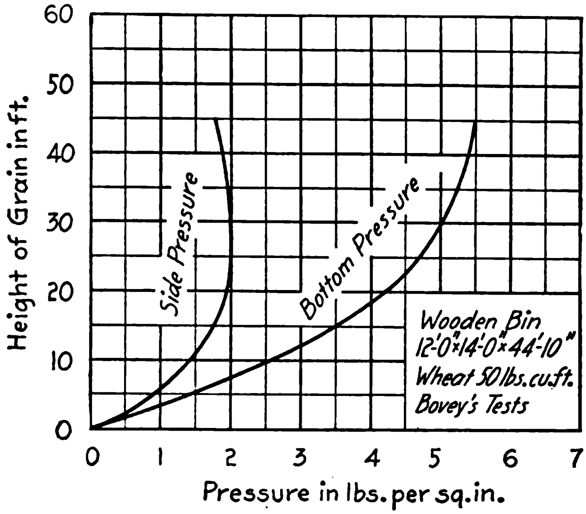

FIG. 173. LATERAL AND VERTICAL PRESSURE, BIN FILLING. DIAPHRAGM (*B*) AT CENTER OF BIN.

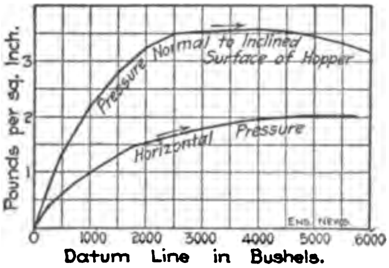

FIG. 174. LATERAL AND VERTICAL PRESSURES, BIN FILLING. DIAPHRAGM (*B*) ON HOPPER SIDE.

The pressures in Fig. 174 were obtained with diaphragm (*C*), placed as before, and diaphragm (*B*) placed parallel to and half-way down the slant of the hopper. All pressures were referred to the top of the hopper. Pressures were determined with the grain in motion, and the effect of a sudden stoppage after each 500 bushels was determined. The sudden stopping produced no appreciable effect. The maximum vertical pressure was 3.592 lbs. per sq. in., as compared with a maximum of 5.654 lbs. per sq. in. with diaphragm (*B*) at the center. The grain was left in the elevator over night, the pressure having slightly decreased. The bin was emptied, observations being made at intervals of ½ minute and 1 minute. The pressures and the effect of opening and closing the gates are shown in Fig. 175.

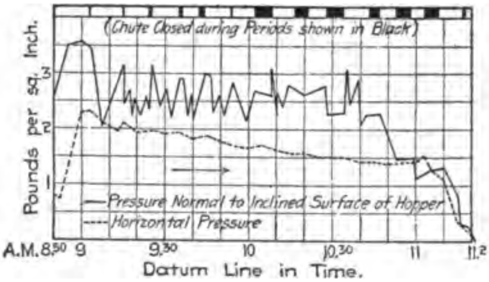

FIG. 175. LATERAL AND VERTICAL PRESSURES, BIN EMPTYING, DIAPHRAGM ON INCLINED HOPPER SURFACE AND ON BOTTOM OF BIN WALL.

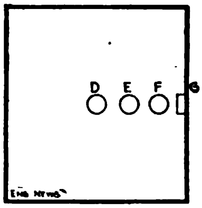

FIG. 176. LOCATION OF DIAPHRAGMS. (FIG. 5.)

Series V. Great Northern R. R. Elevator, Quebec.—The bin used was a timber crib bin, 13.4 × 12.35 ft. in cross-section, with a hopper gate 14 inches in diameter. The hopper was filled to the top and the diaphragms were placed as in Fig. 176. Diaphragms *D, E,* and *F* had a sectional area of 26 sq. in. each and were placed horizontal, while diaphragm *G* had a sectional area of 109.36 sq. in., and was placed vertical and parallel to the face of the bin. The diaphragms were calibrated before and after the experiments.

The pressures as given by the four diaphragms, readings being taken at intervals of 20 seconds, are shown in Fig. 177. The pressures on the diaphragms *D, E,* and *F* show that the pressures diminish from the center to the sides of the bins, and corroborate the theory that the grain in the bin consists of a series of superimposed domes. It will be noted that the lateral pressures determined by the large diaphragm on the Canadian Pacific Ry. elevator correspond very closely with the pressures determined by the small diaphragm, indicating that the size of the diaphragm has no appreciable effect on the pressure. Opening and closing the gates produced no appreciable effect on the pressures.

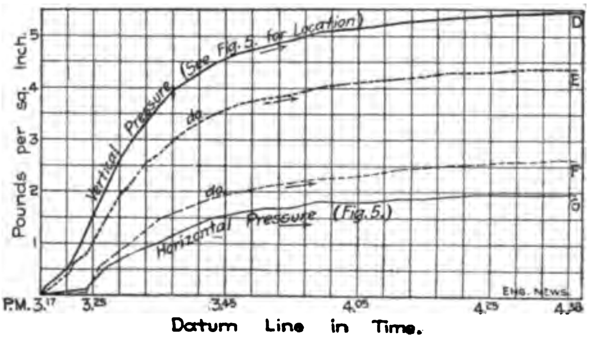

FIG. 177. LATERAL AND VERTICAL PRESSURES, BIN FILLING.

LUFFT'S EXPERIMENTS.*—Eckhardt Lufft made a series of experiments to determine the pressure of grain on full-sized bins in 1902 and 1903, at Buenos Aires, Argentina. These experiments were made before Jamieson's experiments were published. The tests were made on two bins, one 23′ 10″, and the other 11′ 3″ diameter, both bins being 54.8 feet deep, arranged as shown in Fig. 178. The inside surface of the bins was faced with rich cement mortar.

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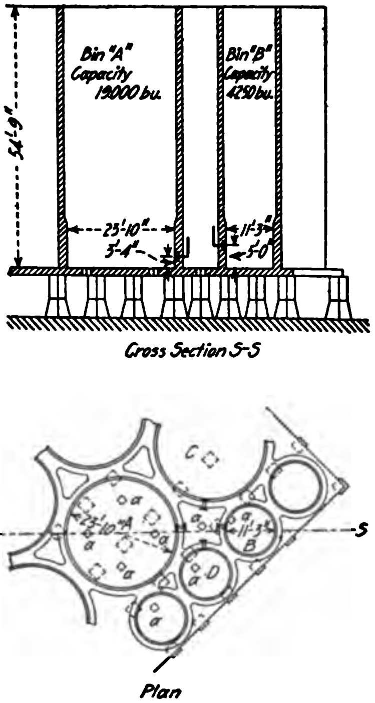
* *Eng. News*, 1904, Vol. 52, p. 531.

The pressure gage used in making the tests is shown in Fig. 179. The tests were made on bins *A* and *B*, the pressure gages being located as shown in Fig. 178. The bin discharge gates are marked *a*.



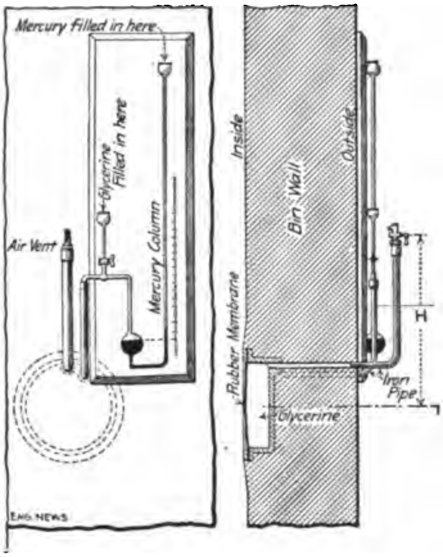


FIG. 178. PART PLAN OF BUENOS AIRES GRAIN ELEVATOR SHOWING LOCATION OF PRESSURE GAGES.

FIG. 179. PRESSURE GAGE.

The pressures of wheat on bin *A*, taken May 4, 1903, for the bin filling and emptying are given in Fig. 180. The grain was conveyed to the bin with a belt conveyor, and was weighed in by an automatic scale. Readings of the gage were taken at intervals of from ten to fifteen minutes. If the outlet nearest the gage was rapidly opened the pressure was decreased, while if the outlet was opened very slowly there was a slight pressure increase. Moving grain made no appreciable increase in the pressure.

The pressure curves for filling and emptying corn weighing 48.4 lbs. per cubic foot are given in Fig. 181. The broken filling curve is due to the fact that the filling was interrupted. In this case the pressure decreased for several hours or days before it became constant. It would appear that this decrease in pressure is due to the settlement of the grains, so that both the angle of repose and the coefficient of friction of the grain on the bin walls are increased.

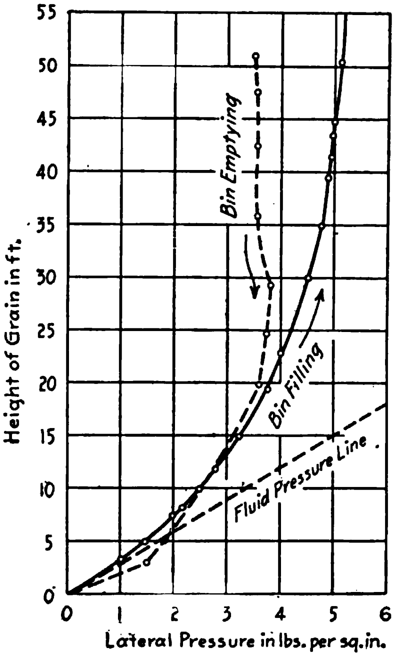


FIG. 180. WHEAT BIN FILLING AND EMPTYING. WHEAT 48 LBS. PER CU. FT., SMOOTH CONCRETE BIN 23′ 10″ DIAMETER.

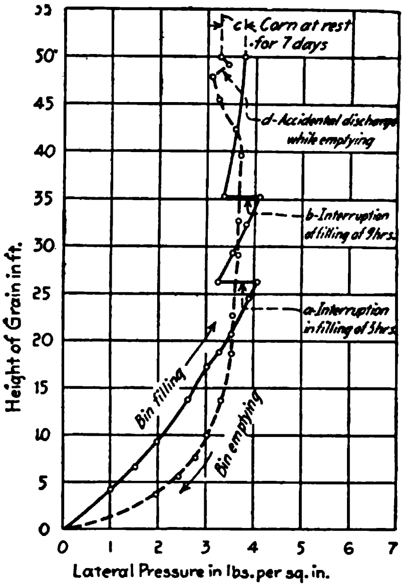


FIG. 181. CORN BIN FILLING AND EMPTYING. CORN 48.4 LBS. PER CU. FT., SMOOTH CONCRETE BIN 23′ 10″ DIAMETER.

The pressure curves for filling the small bin with wheat of different grades are given in Fig. 182. A typical emptying curve for the same bin is given in Fig. 183. In this bin the gage was almost vertically above the outlet gate, and was exposed alternately to the pressure of flowing grain and the pressure of grain at rest.

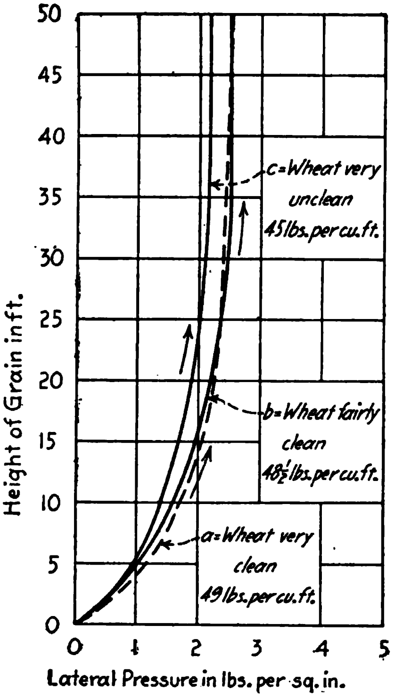


FIG. 182. WHEAT BIN FILLING, DIFFERENT GRADES OF WHEAT, SMOOTH CONCRETE BIN 11′ 3″ DIAMETER.

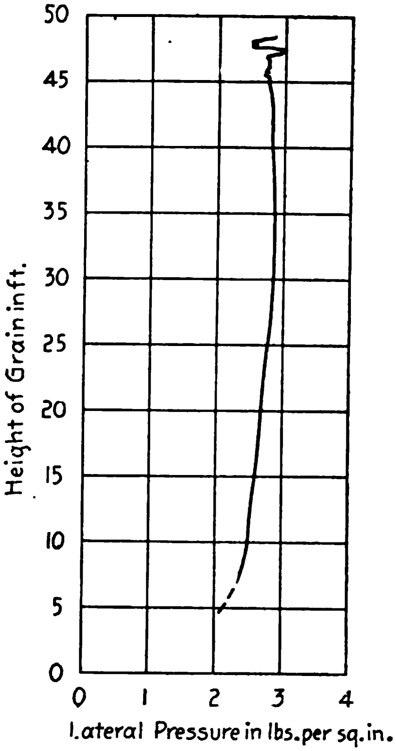


FIG. 183. WHEAT BIN EMPTYING, SMOOTH CONCRETE BIN 11′ 3″ DIAMETER.

PLEISSNER'S EXPERIMENTS.*—In 1902–1905 Mr. J. Pleissner made an extensive series of experiments on the pressure of grain in deep bins for the firm of T. Bienert, in Dresden-Plauen, Germany.

Bins.—The experiments were made (*a*) with a cribbed timber bin, 1.57 meters square and 18 meters deep; (*b*) with a cribbed timber bin, 1.57 meters square and 18 meters deep, with rings spaced 1.6 meters apart vertically and projecting 45 mm. into the bin; (*c*) with a small plank bin with planks vertical, 1.5 meters square and 18 meters deep;

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* "Versuche zur Ermittlung der Boden und Seitenwanddrucke in Getreidesilos." *Zeitschrift des Vereines deutscher Ingenieure,* June 23, 30, 1906, p. 976.

(*d*) with a large plank bin with planks vertical, 2.15 meters × 2.9 meters in section and 9 meters deep; and (*e*) with a reinforced concrete bin 2.5 meters × 3.15 meters in section and 17.24 meters deep, with rings spaced 2.93 meters apart vertically and projecting 80 mm. into the bin.

The surfaces of the cribbed bins (*a*) and (*b*) were quite rough, the planks in the plank bins (*c*) and (*d*) were unplaned, while the concrete in the reinforced concrete bin (*e*) had been filled and tamped against rough board forms which left a very rough surface. The coefficients of friction μ' on the bin wall surfaces for different grains are given in Table XXXVII.

TABLE XXXVII.

COEFFICIENT OF FRICTION OF GRAIN ON BIN WALLS.

Bins.	Coefficient of Friction $\mu' = \tan \phi'$.	
	Wheat.	Rye.
(*a*) Cribbed bin	0.43	0.54
(*b*) Ringed cribbed bin	0.58	0.78
(*c*) Small plank bin	0.25	0.37
(*d*) Large plank bin	0.45	0.55
(*e*) Reinforced concrete bin	0.71	0.85

Pressure Measuring Apparatus. *Bottom Pressures.*—The bottom pressures in bins (*a*), (*b*), and (*c*) were determined by weighing the pressure directly by supporting the movable bottom by means of a rod passing through the center of the bin to a scale beam above. The bottom pressures in the other bins were measured by observing the deflections of the pitch pine plank bottoms, the pressures being later determined from actual experiments on the deflections of the planks. The planks in the bottom of the reinforced concrete and large plank bins were 300 mm. (12 in.) wide, 90 mm. (3.6 in.) thick, with a clear span of 2.5 meters (8.2 ft.). The bottom pressures were also determined by means of two rubber bags filled with water, placed on the plank bottoms and connected with glass scale tubes.

Side Pressures.—The side pressures were determined by measuring the deflections of plank pressure surfaces, made of planks placed vertically, the pressures being later determined from actual experiments.

The pressure surfaces for (*e*), the reinforced concrete, and (*d*), the large plank bin, were each 3.15 m. (10.33 ft.) wide and 2.5 m. (8.2 ft.) high, and were made of planks 300 mm. (12 in.) wide, 70 mm. (2.8 in.) thick, with a clear span of 2.5 m. (8.2 ft.). The pressure measuring surface in each case had the lower edge even with the bottom and extended across an entire side of the bin. The side pressure surfaces in bins (*a*), (*b*), and (*c*) were 1.375 m. (4.5 ft.) wide, by 0.9 m. (2.95 ft.) high, and were made of seven planks 195 mm. (7.8 in.) wide, 25 mm. (1 in.) thick, with a clear span of 860 mm. (2.82 ft.). The side pressures were also determined by means of three rubber diaphragms filled with water and placed on a side next to pressure surface, with their centers at the same height as the center of the plank pressure surface and 560 mm. apart. The sizes of the rubber diaphragms were not given.

The Tests.—The bottom and side pressures were determined for wheat, rye, flax-seed, and rape, with the grain at rest and in motion. The tests were repeated until results that agreed within themselves and with each other were obtained. In all 126 separate tests were made from 1902 to 1905.

Grain at Rest.—The lateral and vertical pressures of wheat at rest in the reinforced concrete bin (*e*) are given by the full lines in Fig. 184. These pressures were determined by means of the plank pressure surfaces. The pressures calculated by Janssen's formulas (144) and (145) with $w=50$ lbs. per cu. ft., $\mu'=0.71$, and. $k=0.3$, are shown by dotted lines. The calculated pressures agree almost exactly in the case of the lateral pressures, and the agreement with the vertical pressures is very close.

The lateral and vertical pressures of wheat at rest in the small plank bin (*c*) are given by the full lines in Fig. 185. These pressures were determined by means of the plank pressure surfaces. The pressures calculated by Janssen's formulas (144) and (145) with $w=50$ lbs. per cu. ft., $\mu'=0.25$, and $k=0.34$, are shown by dotted lines. The calculated and experimental values of the lateral pressure agree very closely, while the vertical pressures show that k was a variable.

Grain in Motion.—Tests were made to determine the lateral and vertical pressures with the grain in motion. The results with the rein-

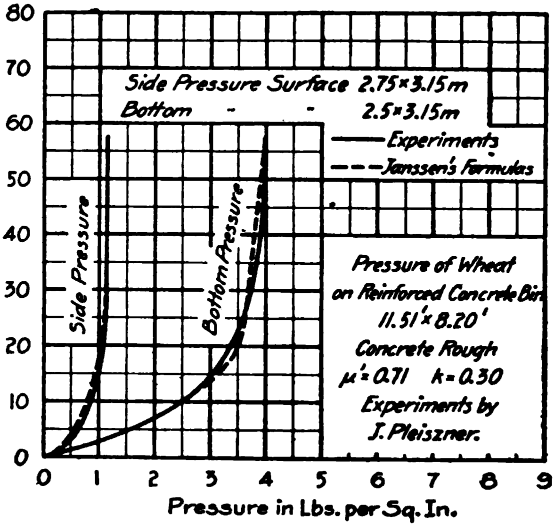

FIG. 184.

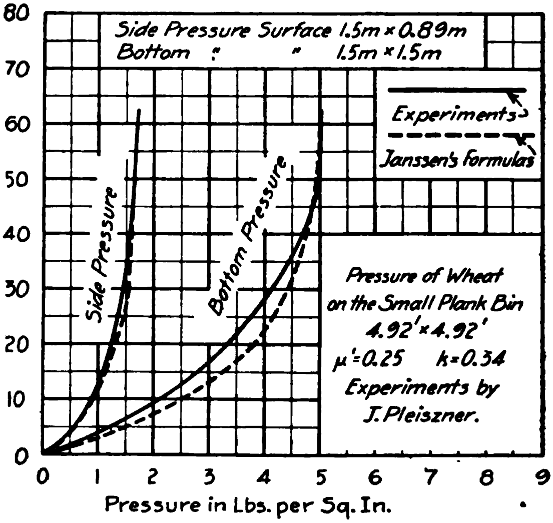

FIG. 185.

forced concrete bin only will be discussed. The vertical pressures, as determined by the rubber diaphragms or the deflection of the plank bottoms, showed no appreciable increase with the grain in motion, the average velocity of the grain in the bin being approximately 1 mm. per second, the same as in Prante's experiments.

The lateral pressures, as determined by the plank pressure surfaces, were much increased, while the lateral pressures, as determined by the rubber diaphragms, were erratic; the rubber diaphragm 1 placed midway between the gate and the plank pressure surface showing nearly double the static pressure; rubber diaphragm 3, placed 280 mm. from the side opposite the plank pressure surface, showed only a very slight increase over the static pressure; while the rubber diaphragm 2, placed midway between diaphragms 1 and 3, gave pressures between those given by 1 and 3.

The discharge gates were located in the bottom, one near the center and the other near the side opposite the plank pressure surface. This explains the eccentricities of the pressures indicated by the rubber diaphragms and the increase in pressure on the plank pressure surface opposite the gates; for Fig. 165 and the other experiments show that, with an opening in the bottom of the side of a bin, the pressures due to moving grain are materially increased on the side opposite the gate and slightly decreased on the side in which the gate is placed. Pleissner's experiments confirm the above conclusions, but are of little value in determining the question of increase in lateral pressure due to moving grain, as raised by Prante. To investigate the variation of side pressures for grain in motion the discharge gate must be in the center of the bin. Grain bins should never be constructed with side discharge gates.

A very interesting phenomenon was that the observed increase in pressure for grain in motion was very much less with rye than with wheat, the velocity in the bin being the same.

Value of k, the Ratio of Lateral to Vertical Pressures.—Pleissner found that *k*, the ratio of the lateral to vertical pressures, was not a constant for a grain in a bin at different depths, being greater for small than for large depths of grain, and that it varies with the different bins and different grains. Average values of *k* as determined for wheat and rye in the different bins are given in Table XXXVIII.

TABLE XXXVIII.

VALUES OF $k=\frac{L}{V}$ FOR WHEAT AND RYE IN DIFFERENT BINS.

Bins.	$k=\frac{L}{V}$	
	Wheat.	Rye.
(*a*) Cribbed bin	0.4 to 0.5	0.23 to 0.32
(*b*) Ringed cribbed bin	0.4 to 0.5	0.3 to 0.34
(*c*) Small plank bin	0.34 to 0.46	0.3 to 0.45
(*d*) Large plank bin	0.3	0.23 to 0.28
(*e*) Reinforced concrete bin	0.3 to 0.35	0.3

Rape and flax-seed were tested in the small plank bin (*c*) and gave values of $k=0.5$ to 0.6 for rape, and $k=0.5$ to 0.55 for flax-seed.

DISCUSSION OF EXPERIMENTS.—Since the publication of Prante's experiments in 1896 there has been a wide difference of opinion among engineers on (1) the effect of the motion of grain on the lateral pressure, and (2) the effect of the size of the pressure surface on the indicated pressures.

1. Prante obtained lateral pressures four times the static pressures with wheat moving with a velocity of 1 mm. per second, and decided that with increasing velocities the lateral pressures rapidly approach the full hydraulic pressure as a limit. Pleissner obtained lateral pressures twice the static pressures with wheat moving with a velocity of 1 mm. per second. Jamieson found an increase in the pressure of moving wheat of only 4 to 7 per cent. Bovey found an increase of 9.7 per cent in the Canadian Pacific Ry. Grain Elevator. Lufft and the author found no appreciable increase of the pressure of moving grain over the static pressure.

In the experiments of Prante and Pleissner the bin discharge gates were not located in the center of the bins. These experiments therefore show the distorting effect of an eccentric discharge gate, as is evident from a study of Fig. 165, Fig. 166, Bovey and Lufft's experiments, and give no definite information as to the effect of moving grain on the lateral pressure. The experiments made thus far would appear to prove conclusively that with discharge gates in the center of the bottom and a discharge area not greater than 1/150 the area of the bin the increase in pressure due to moving grain will not exceed the static pressure by more than 10 per cent.

The lateral pressure during emptying is less than during filling when the grain is deep, and greater than during filling when the bin is nearly emptied.

2. Pleissner states that the grain arches over a small pressure surface, so that the results of tests with small pressure surfaces do not give absolutely correct results, although they show relative changes correctly. He drew this inference from the readings on the three rubber diaphragms used to determine lateral pressures. The variations in the pressures indicated by the rubber diaphragms were certainly due to the eccentric discharge gate. It will be seen in Fig. 184 and Fig. 185 that Pleissner's tests on grain at rest with large pressure surfaces agree very closely with the pressures calculated by Janssen's formulas. Experiments by Bovey, Jamieson and the author show that the arching action is insignificant, and that the pressures obtained with small and large pressure surfaces give a remarkably close agreement. The results obtained with model bins agree very closely with the results obtained with full-sized bins.

Conclusions.—The following conclusions may be drawn from the foregoing experiments:

1. The pressure of grain on bin walls and bottoms follows a law (which for convenience will be called the law of "semi-fluids"), which is entirely different from the law of the pressure of fluids.

2. The lateral pressure of grain on bin walls is less than the vertical pressure (0.3 to 0.6 of the vertical pressure, depending on the grain, etc.), and increases very little after a depth of 2½ to 3 times the width or diameter of the bin is reached.

3. The ratio of lateral to vertical pressures, *k*, is not a constant, but varies with different grains and bins. The value of *k* can only be determined by experiment.

4. The pressure of moving grain is very slightly greater than the pressure of grain at rest (maximum variation for ordinary conditions is probably 10 per cent).

5. Discharge gates in bins should be located at or near the center of the bin.

6. If the discharge gates are located in the sides of the bins, the lateral pressure due to moving grain is decreased near the discharge gate and is materially increased on the side opposite the gate (for common conditions this increased pressure may be two to four times the lateral pressure of grain at rest).

7. Tie rods decrease the flow but do not materially affect the pressure.

8. The maximum lateral pressures occur immediately after filling, and are slightly greater in a bin filled rapidly than in a bin filled slowly. Maximum lateral pressures occur in deep bins during filling.

9. The calculated pressures by either Janssen's or Airy's formulas agree very closely with actual pressures.

10. The unit pressures determined on small surfaces agree very closely with unit pressures on large surfaces.

11. Grain bins designed by the fluid theory are in many cases unsafe as no provision is made for the side walls to carry the weight of the grain, and the walls are crippled.

12. Calculation of the strength of wooden bins that have been in successful operation shows that the fluid theory is untenable, while steel bins designed according to the fluid theory have failed by crippling the side plates.

CHAPTER XVIII.

THE DESIGN OF GRAIN BINS AND ELEVATORS.

TIMBER GRAIN ELEVATORS.—The following description and specifications for the timber of the working house of the Missouri Pacific Ry. Elevator, Kansas City, Mo., will give a clear idea of timber elevator construction. For a view of this elevator see Fig. 148. The elevator was designed by the John S. Metcalf Co., Chicago, Ill.

General Description.—The working house is a plank and frame structure 71 ft. wide × 120 ft. long. The first story is built of heavy post and girder work; on top of this are the bins, 45 feet deep, made of laminated 2″ planking, and the bins are surmounted by a cupola five stories in height. The walls are covered with galvanized corrugated steel, and the roof is covered with a tar and gravel roofing. The foundation of the working house is a monolithic concrete slab reinforced with steel rods.

Specifications for Timber and Lumber.—All timber shall be yellow pine, furnished rough. All long leaf yellow pine shall be cut from live, unbled, long leaf pine grown south of Chattanooga. It shall be full size and square-edged, free from all unsound or black knots, splits or shakes. In no case shall wane measure more than 1½″ measured diagonally or extend more than one-third the length or show on more than one corner of the stick. The sap specification shall be "Mercantile Inspection" under Savannah Rules of February 14, 1883, as follows: "All square timbers shall show two-thirds heart on two sides and not less than one-half heart on two other sides. Other sizes shall show two-thirds heart on faces and show heart two-thirds of the length on edges, except where the width exceeds the thickness by 3 inches or over, then it shall show heart on one-half the edges."

All other yellow pine timber shall be No. 1 common yellow pine, cut from live timber, and shall conform to the specifications for No. 1 common timber as set forth in paragraphs 78, 79, and 80 of the

Southern Lumber Manufacturer's Association Standard Classification, Grading and Dressing Rules for yellow pine as revised and adopted at St. Louis, Mo., July 9, 1902, as follows:

"§ 78. Rough timber 4" × 4", and larger, shall not be more than ¼" scant when green, shall be well manufactured with not less than three square edges, and shall be free from knots that will materially weaken the piece. Timbers 10" × 10" in size may have 2" wane on one corner, measured on faces, or its equivalent on two or more corners, one-third the length of the piece. Larger sizes may have proportionally greater defects. Shakes extending not over one-eighth of the length of the pieces are admissible, and seasoning cracks shall not be considered defects.

"§ 79. Dressed timbers shall conform in grading to the specifications applying to rough timbers of same size.

"§ 80. Rough timbers, if thicker than the specified thickness for dry or green stock, may be dressed to such standard thickness, and when so dressed, shall be considered as rough stock."

Bin Planking.—The bin planking shall be thoroughly dry yellow pine, dressed on one side to 1⅝" and on one edge to the specified width. It shall conform to the specifications for No. 1 common dimension as specified in paragraph 66 of the Standard Classification, Grading and Dressing Rules for yellow pine as revised and adopted by the Southern Lumber Manufacturer's Association at St. Louis, Mo., July 9, 1902, as follows: "No. 1 common dimension may contain sound knots, none of which in 2" × 4" should be larger than 2" in diameter on one or both sides of the piece, and on wider stock which do not occupy more than one-third of the cross-section at any point throughout its length if located at the edge of the piece, or more than one-half the cross-section if located away from the edge; two pits, knots, or smaller or more defective knots which do not weaken the piece more than the knots aforesaid; will admit of seasoning checks; firm red heart, heart shakes which do not go through, wane, pitch, blue sap stains, pitch pockets, splits in the ends not exceeding in length the width of the piece, a limited number of small pin-worm holes well scattered, and such other defects as do not prevent its use as substantial structural material."

Bins.—The bins shall be constructed of planking in courses laid as follows: All walls shall be laid up plumb and true, forming square corners in the bins. Each course of planking shall be securely nailed with 30-d wire nails 4½" long, said nails not exceeding 22 to the pound.

At all interior crossings two nails shall be driven in each end of each piece of plank in the 4″ and 6″ walls. At all outside crossings five nails shall be driven in each end of each plank crossing the wall and four nails in the plank at right angles to it. The intermediate nails shall be staggered. At the base of the walls and for 11′ 0″ in height the nails shall be spaced not to exceed 14″ centers, and for each additional 11′ 0″ in height the distance apart may be increased 2″, making the spacing in the top 11′ 0″ of planking 20″ centers. The heads of the nails shall be well bedded in the wood.

The floors shall consist of two thicknesses, the first layer to be $\frac{7}{8}$″ shiplap, the second layer to be 2″ × 6″ dressed on one side and matched through the trackway, and the second layer over the balance of the floor shall be 1″ × 6″ flooring. All flooring shall be thoroughly dry yellow pine to conform to "C" flooring.

The floor of the engine house shall have the bottom thickness of $\frac{7}{8}$″ shiplap and the top thickness of 1″ × 2″ dressed and matched hard maple flooring.

Allowable Stresses in Timber.—The timber parts of grain elevators should be proportioned in accordance with the following unit stresses, given in pounds per square inch:

Kind of Timber.	Transverse Loading.	End Bearing.	Columns Under 10 Diameters. C	Bearing Across Fiber.	Shear Along Fiber.
White Oak..................	1,200	1,200	1,000	500	200
Long Leaf Yellow Pine...	1,500	1,500	1,000	350	100
White Pine and Spruce....	1,000	1,000	600	200	100
Hemlock	800	800	500	200	100

Columns may be used with a length not exceeding 45 times the least dimension. The unit stress for lengths of more than 10 times the least dimension shall be reduced by the following formula:

$$P = C - \frac{C}{100} \cdot \frac{l}{d}$$

Where C = unit stress, as given above for short columns;

P = allowable unit stress, in lbs. per sq. in.;

l = length of column, in inches;

d = least side of column, in inches.

CIRCULAR STEEL BINS.—In the designing of steel grain bins particular attention should be given to the horizontal joints, and to the strength of the bin to act as a column to support the grain. To calculate the thickness of the metal the horizontal pressure L is obtained from Janssen's or Airy's formulas, Chapter XVI, and then the thickness will be found by the formula

$$t = \frac{L.d}{2S.f} \qquad (153)$$

where t = thickness of the plate in inches,
L = horizontal pressure in lbs. per sq. in.,
d = diameter of bin in inches,
S = working stress in steel in lbs. per sq. in.,
f = efficiency of the joint.

The unit stress S may be taken at 16,000 lbs. per square inch, and f will be about 57 per cent for a single riveted lap joint, 73 per cent for a double riveted lap joint, and 80 per cent for double riveted double strap butt joints.

The allowable stresses given in the author's "The Design of Steel Mill Buildings" should be used in design. These allowable stresses are as follows: Tension on net section 16,000 lbs. per sq. in.; shear on cross-section of rivets 11,000 lbs. per sq. in.; bearing on the projection of rivets (diameter × thickness of plate) 22,000 lbs. per sq. in. Compression in columns $P = 16{,}000 - 70\frac{l}{r}$ where P = unit stress in lbs. per sq. in., l = length of member and r = radius of gyration of the member, both in inches.

Rivets in Horizontal Joints.—The side walls carry a large part of the weight of the grain in the bin and this should be considered in designing the horizontal joints. The weight of the grain supported by the bin above any horizontal joint can be calculated as shown in the following example: Assume a steel plate bin 25 ft. in diameter, and it is required to calculate the grain supported by the bin walls above a horizontal joint 75 feet below the top of the grain. We will use the diagram for wooden bins in Fig. 157. In Fig. 157 we enter the table with $h/d = 3$, and find $P/d^2 = 60$. This gives the total pressure on each lineal foot of circumference = 37,500 lbs. Multiplying by ⅘, we have 30,000 lbs. as the pressure on a smooth plate bin. The coefficient of friction of grain on a smooth steel bin is $\mu' = 0.375$, and the weight of grain supported by a lineal foot of the steel plate bin wall will be = 30,000 × 0.375 = 11,250 lbs. The weight of the steel bin above the joint may be taken as 1,250 lbs. per lineal foot of joint. The horizontal riveting should then be designed for a shear of 12,500 lbs. per lineal foot of joint. Assume that the plates are ⅜″ thick and the rivets ¾″ in diameter. For allowable stresses of 16,000 lbs. per sq. in. in tension, 11,000 lbs. per sq. in. in shear, and 22,000 lbs. per sq. in. in compression, we have from Table XXVIII the value of a ¾″ shop rivet in single shear = 4,860 lbs., and a field rivet is ⅔ of 4,860 = 3,240 lbs., and in compression = 6,190 lbs. for shop rivets and = 4,127 lbs. for field rivets. For a lap joint therefore the spacing should not be greater than 3,240 × 12 ÷ 12,500 = 3⅛″. *

Stiffeners.—In large circular steel bins the thin side walls are not sufficiently rigid to support the weight of the grain and it is necessary to supply stiffeners. For this purpose angles or Z-bars may be used. Experience has shown that bins in which the height is equal to or greater than about 2½ times the diameter do not need stiffeners. There

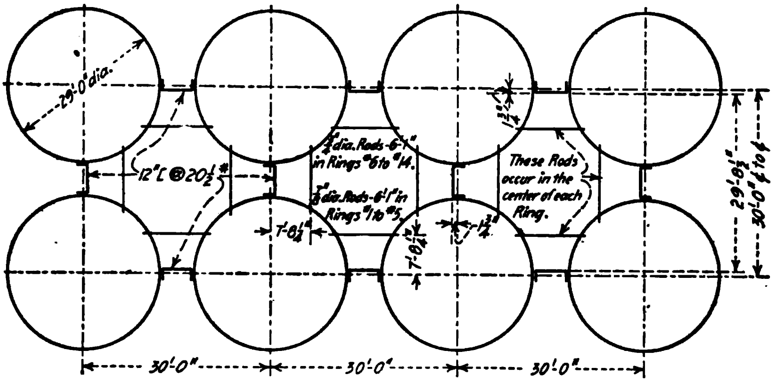
29′-0″ dia.
12″ [@ 20½#
¾″ dia. Rods-6′-1″ in Rings #6 to #14.
⅞″ dia. Rods-6′-1″ in Rings #1 to #5.
These Rods occur in the center of each Ring.
1′-3¾″
7′-8¼″
7′-8¼″
1′-1¾″
29′-8½″
30′-0″ ℄ to ℄
30′-0″
30′-0″
30′-0″

FIG. 187. PLAN OF STEEL STORAGE BINS FOR A STEEL ELEVATOR.

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* For stresses due to wind moment, see page 311.

is at present no rational method for the design of these stiffeners or the stiffeners in plate girders. In Fig. 210 will be seen the details of a steel bin of the Independent Steel Elevator with Z-bar stiffeners. Angle stiffeners were used in the bins of the Electric Elevator, Minneapolis, Minn.

Circular steel bins are used for storage in large elevators and may be used for a complete elevator as in Fig. 187. The space between the bins is sometimes used for auxiliary storage as in Fig. 187. The circular bin walls are stiffened by means of vertical channels, and the auxiliary bins are cross-braced with steel rods. Complete details of circular steel bins for the Independent Elevator, Omaha, Neb., are shown in Fig. 210.

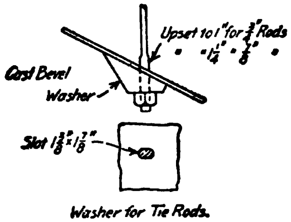

FIG. 188. DETAIL OF THE ROD CONNECTION IN FIG. 187.

Steel Country Elevator.—General plans of a steel grain elevator for the Manhattan Milling Co., designed and constructed by the Minneapolis Steel & Machinery Co., Minneapolis, Minn., are given in Fig. 189. This elevator could easily be changed to a shipping elevator by putting in a wagon dump. Grain is run from the cars into the boot of the receiving leg, and is then elevated and conveyed by a screw conveyor to the large storage bins, or is run into the temporary storage bins, then cleaned and elevated and conveyed to the storage bins by the screw conveyor. The bins are built of steel plates, and the working house is built of steel framework covered with corrugated steel. This elevator has a capacity of 76,300 bushels but the scheme can be used for a 30,000 to 40,000 bushel elevator for either shipping or for milling purposes.

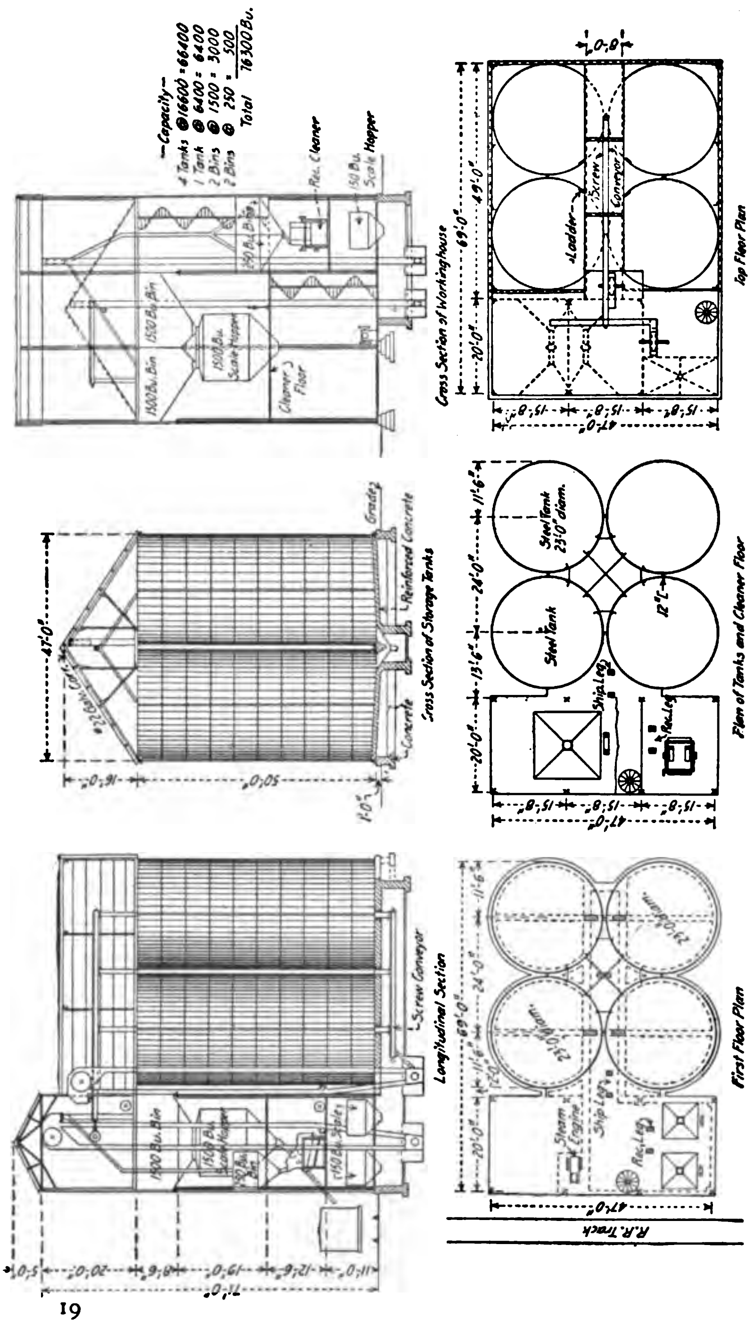

FIG. 189. STEEL ELEVATOR FOR MANHATTAN MALTING CO., MANHATTAN, MONTANA.

RECTANGULAR STEEL BINS.—In designing rectangular bins the thickness of the plates may be found by the diagram for flat plates in Fig. 86. A common size of rectangular bins in grain elevators is about 14 feet square, and in bins of this size it is either necessary to brace the sides or use very heavy plates. Buckle plates are sometimes used for rectangular grain bins.

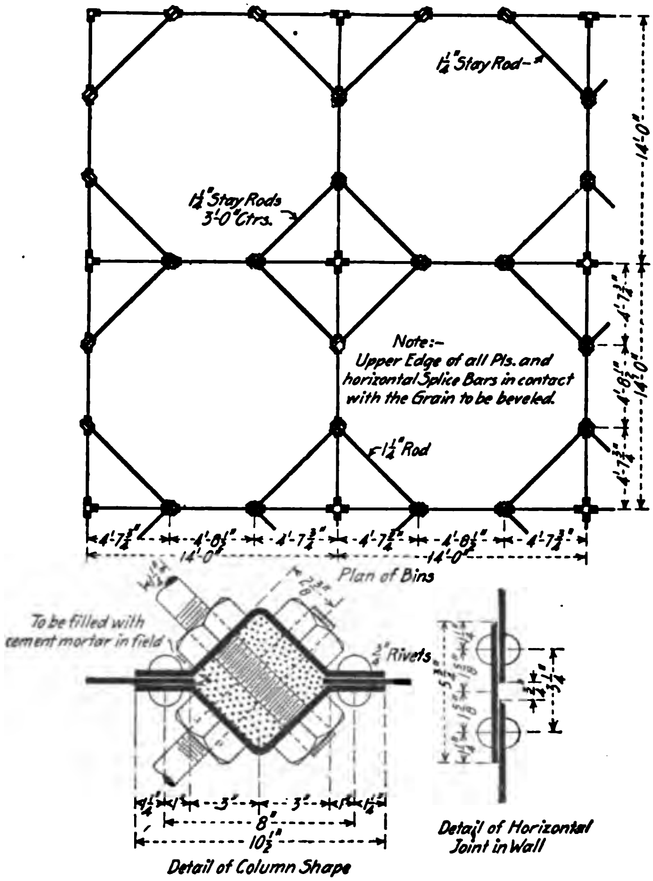
1¼" Stay Rod
1¼" Stay Rods 3'-0" Ctrs.
Note:- Upper Edge of all Pls. and horizontal Splice Bars in contact with the Grain to be beveled.
1¼" Rod
4'-7¾"
4'-8½"
14'-0"
Plan of Bins
To be filled with cement mortar in field
¾" Rivets
2⅜"
1¼"
3"
8"
10½"
Detail of Column Shape
5¼"
1⅛"
3¼"
Detail of Horizontal Joint in Wall

FIG. 190. PLAN OF METCALF RECTANGULAR STEEL BINS.

Metcalf Bin.—The method of constructing rectangular bins shown in Fig. 190 and Fig. 191 has been patented by the John S. Metcalf Co., Chicago. This construction is very economical and satisfactory. The concrete filling makes a continuous column, while the round brace rods are so rigid that the moving grain does not affect them. The Windmill Point steel grain elevator, Fig. 192 and Fig. 193, is an excellent example of an elevator built according to this system.

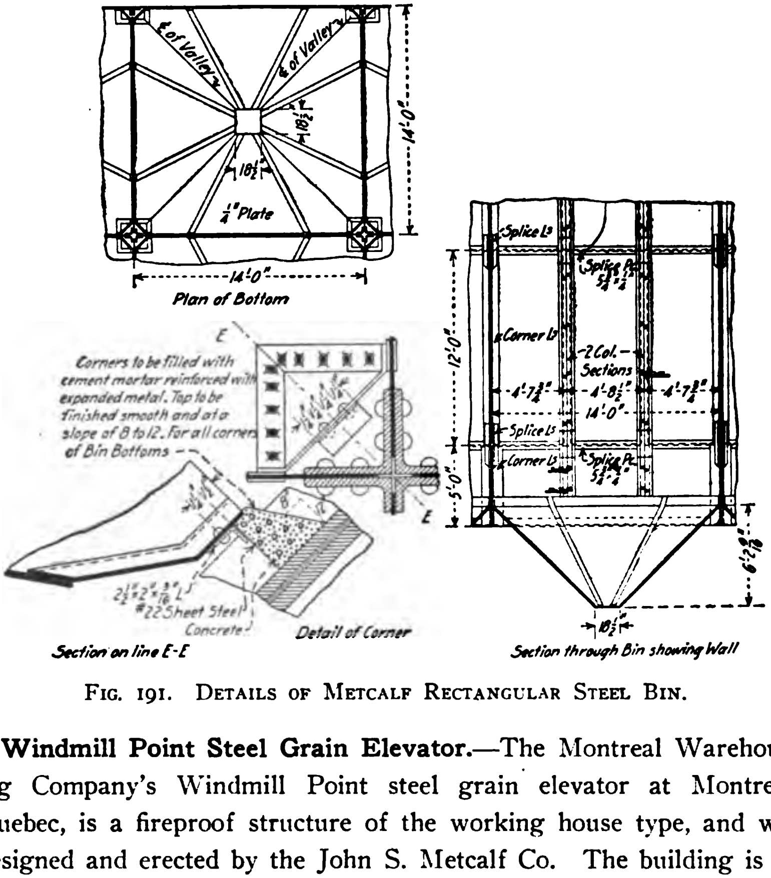

FIG. 191. DETAILS OF METCALF RECTANGULAR STEEL BIN.

Windmill Point Steel Grain Elevator.—The Montreal Warehousing Company's Windmill Point steel grain elevator at Montreal, Quebec, is a fireproof structure of the working house type, and was designed and erected by the John S. Metcalf Co. The building is 84 ft. wide × 238 ft. long. The elevator is shown during construction in Fig. 192 and Fig. 193. The framework is a self-supporting steel structure. The outside covering is of concrete to a height of 7 ft. above the base of rail and brick to a height of 124 ft. above the base of rail; while the cupola is covered with tile. The roofs are made of

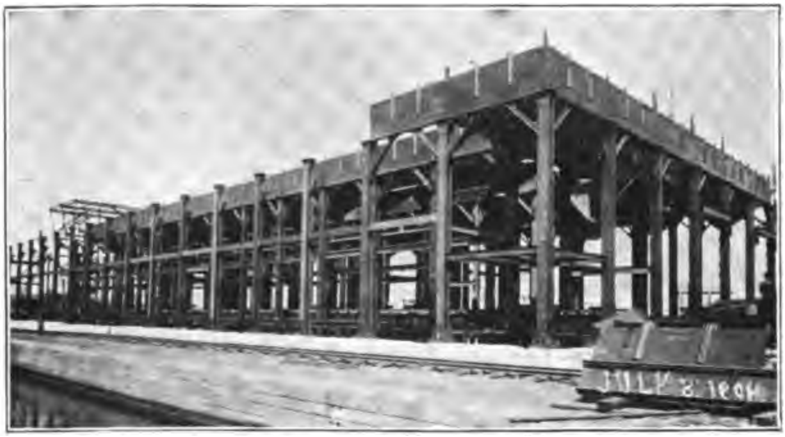

FIG. 192. GRAND TRUNK STEEL GRAIN ELEVATOR, WINDMILL POINT, MONTREAL, QUEBEC.

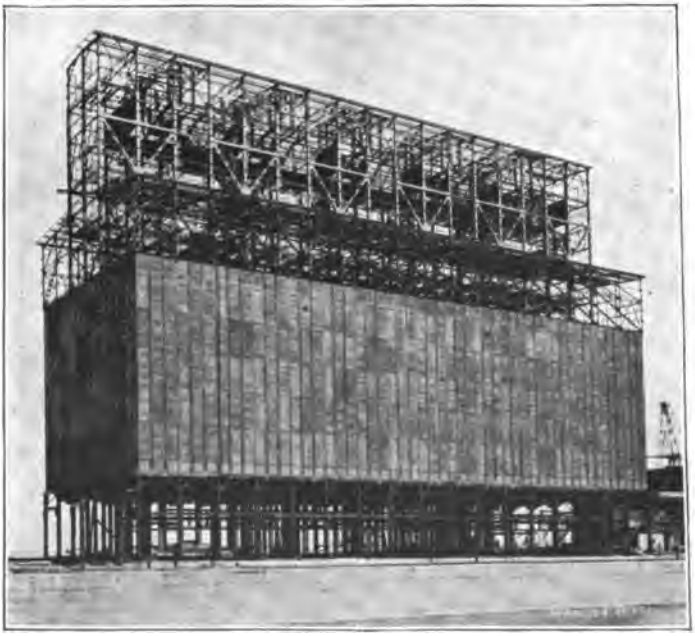

FIG. 193. GRAND TRUNK STEEL GRAIN ELEVATOR, WINDMILL POINT, MONTREAL, QUEBEC.

3″ book tile supported on steel T's and covered with felt, tar and gravel. The floors are of concrete and tile. The windows have metal frames glazed with ¼″ wire glass with mesh of wire netting not larger than 1″. A marine tower constructed of fire-proof materials capable of unloading at the rate of 15,000 bushels per hour is provided. A system of conveyor galleries is provided to carry the grain along the wharf. The conveyor belts are 36″ wide and discharge into dock spouts spaced 60 ft. apart.

The elevator is equipped with 5 receiving and 5 shipping elevator legs, having 20″ × 7″ × 7″ metal cups and 7′ 0″ head pulleys. Six belt conveyors 30″ wide are located in the basement to carry grain across the house and discharge it into the five elevators.

In the first story are located 5 pairs of power shovels for unloading, one car puller with four drums, two elevator separators, and a complete sweeper system. On the west side a 36″ belt brings grain from the marine tower and discharges it into any of the 5 receiving elevators.

In the top story of the cupola ten elevator heads discharge into ten 2,000 bushel garners. Below each garner is a 120,000-lb. hopper scale, surmounted by a 2,000-bushel scale hopper. Below the scale hoppers, in the second story of the cupola, is the spouting to the trolley spouts and two 36″ belt conveyors running the entire length of the cupola. These belts are reversible and are provided with loading spouts and trippers. In the first story of the cupola is located a complete system of trolley spouts for discharging into the bins. For an abstract of the specifications for the steel, see p. 298.

Cross-bracing.—In the Great Northern Elevator the plates were too thin to take the change in loading due to emptying and filling adjacent bins, without undue deflection, and vertical Z-bars were riveted to the middle of the sides of the 13′ 6″ × 13′ 6″ bins. The Z-bars were fastened together with flat steel ties 5″ × ⅜″, with long dimension vertical and riveted at the ends. These ties were torn out and the rivets were sheared off by the moving grain. For details of these rectangular bins see Fig. 221.

In the rectangular bins of the Independent Elevator, Omaha, Neb., Fig. 209, vertical stiffener angles were placed about 5 feet apart in

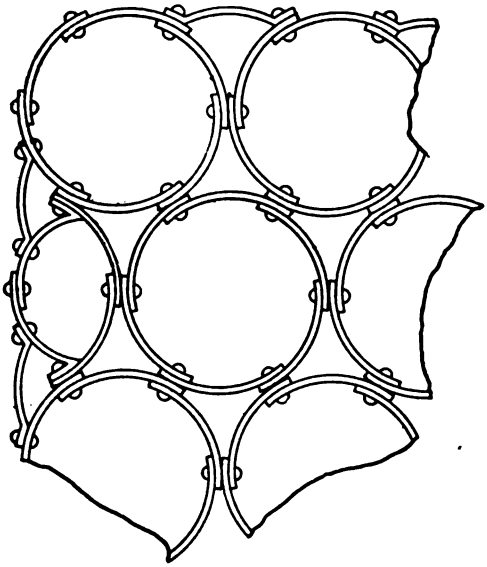

FIG. 194. MACDONALD SYSTEM OF BIN CONSTRUCTION.

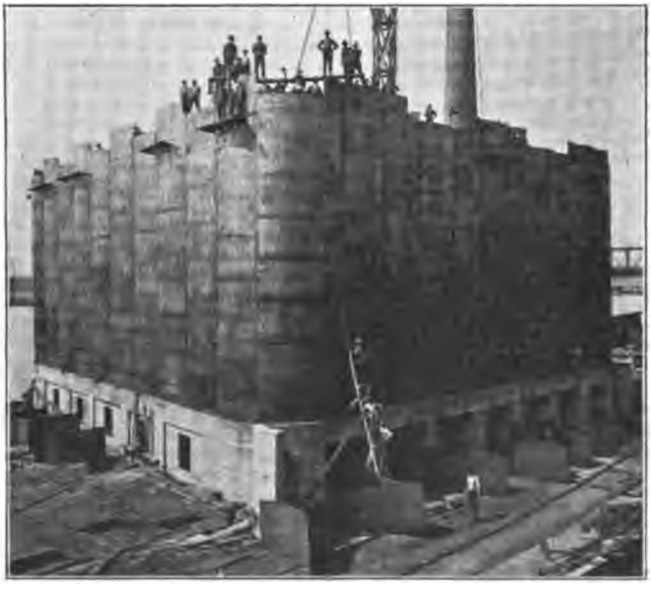

FIG. 195. C. H. & D. ELEVATOR "B," TOLEDO, O. JAMES MACDONALD, ENGINEER.

the bin walls and angle diagonal braces were placed 5 feet apart vertically. The ends of the angles were flattened out under the hammer and were then riveted to bin walls, passing between the wall and the vertical stiffeners. Rod bracing was used to brace the bins. For a complete description of this elevator, with plans and details, see Chapter XIX.

For method of bracing the Metcalf bin see Figs. 190 and 191.

MacDonald Bin.—The system of constructing circular bins in clusters, as shown in Fig. 194, is patented by the MacDonald Engineering Co., Chicago, Ill. This system is used for working house storage bins. Three plates are bolted together on the ground and are then lifted into place and bolted in position, no rivets being used. The horizontal joints are butted and covered with butt straps.

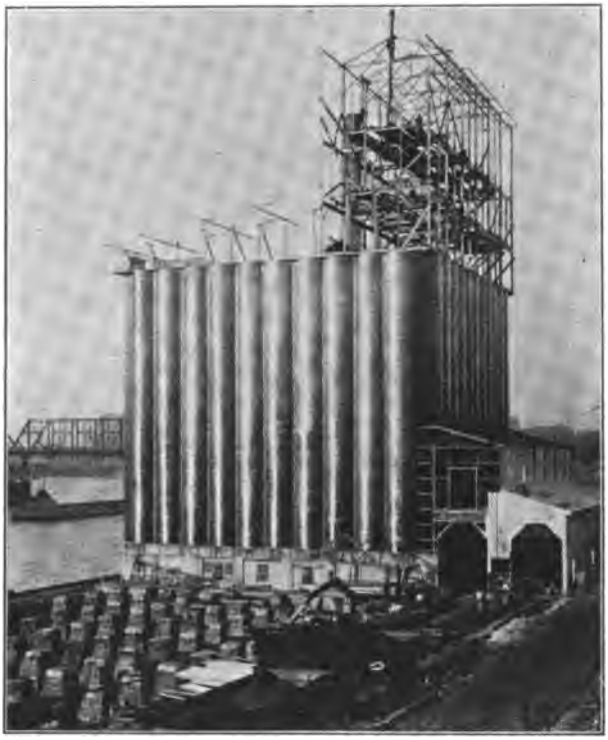

FIG. 196. STEEL GRAIN ELEVATOR CONSTRUCTED ON MACDONALD SYSTEM.

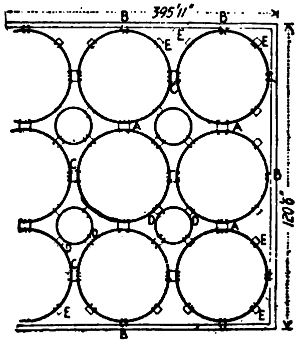

FIG. 197. PLAN SHOWING BIN ARRANGEMENT, GREAT NORTHERN STEEL ELEVATOR, BUFFALO, N. Y.

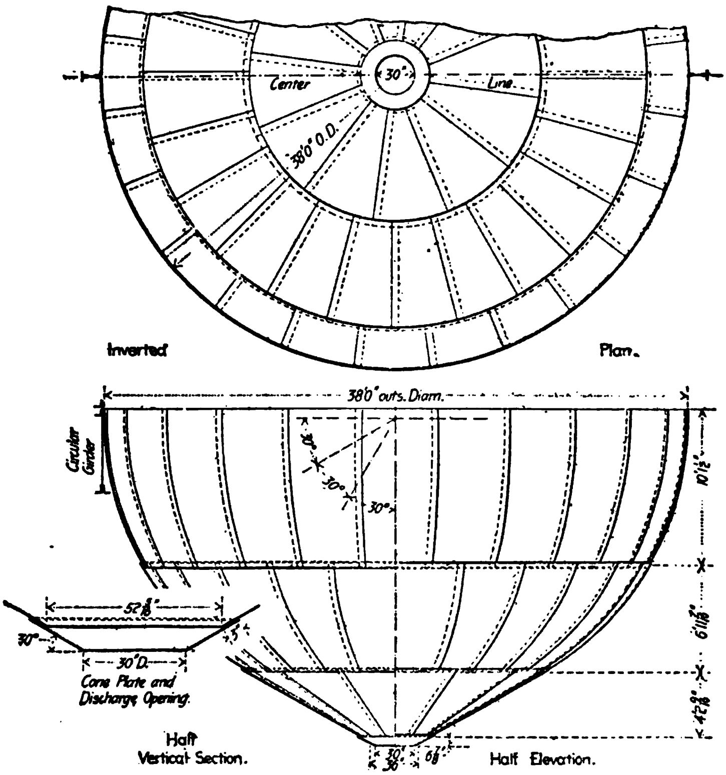

FIG. 198. HEMISPHERICAL BOTTOMS, GREAT NORTHERN STEEL ELEVATOR, BUFFALO, N. Y.

The elevators shown in Fig. 195 and Fig. 196 illustrate the MacDonald system of steel grain elevator construction. A completed elevator is shown in Fig. 145. The MacDonald Engineering Co. has erected many elevators under this system.

The "Great Northern" Steel Elevator, Buffalo, N. Y., designed Max Tolz in 1897, is of the working house type. This elevator has a storage capacity of 2,525,890 bushels. The bins are supported on columns and are of two sizes, 30 bins 38 feet in diameter and 85 feet deep, and 18 bins 15½ feet in diameter, all with hemispherical bottoms. The arrangement of the bins is shown in Fig. 197, while the hemispherical

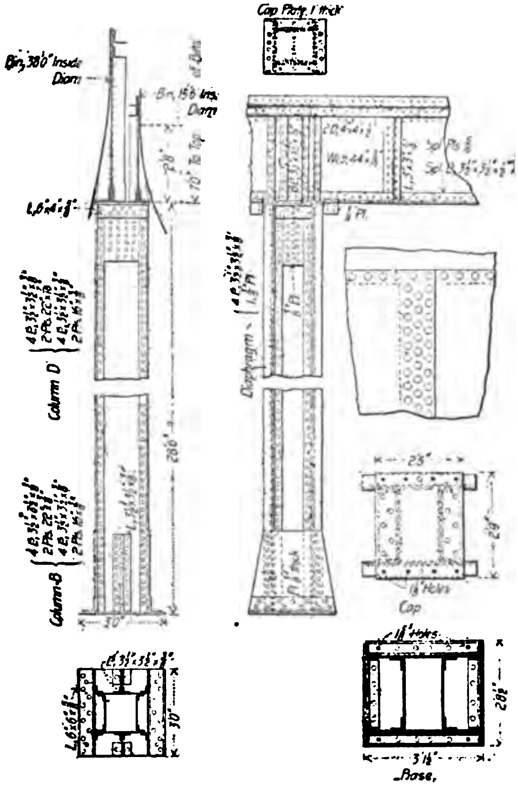

FIG. 199. CIRCULAR GIRDERS AND COLUMNS, GREAT NORTHERN STEEL ELEVATOR, BUFFALO, N. Y.

bottoms are shown in Fig. 198, and the circular girders and columns in Fig. 199. The bins were designed for full fluid pressure, assuming wheat to weigh 50 lbs. per cu. ft. The hemispherical bottoms were made the same thickness as the bottom side plates. The steel work was designed for a safe fiber stress of 17,000 lbs. per square inch in tension. For a complete description see Engineering News, April 7, 1898.

SPECIFICATIONS FOR STEEL WORK.—The specifications for steel grain bins should be the same as for bins, for which see Chapter XI and Appendix II. For specifications for structural steel work see the author's specifications for Steel Frame Mill Buildings, published in "Steel Mill Buildings." For reinforcing concrete soft steel is used with an allowable stress of 16,000 lbs. per sq. in.; and high steel with an ultimate strength of 90,000 to 100,000 lbs. per sq. in. is used with an allowable stress of 25,000 lbs. per sq. in.

The following extracts are from the specifications prepared by the John S. Metcalf Co. for the Windmill Point Steel Grain Elevator. The clauses not given are essentially standard clauses.

Method of Manufacture.—All steel used shall be made by the open hearth process. The bin bottoms, garner bottoms and all bent plates shall be of flange steel; all other plates, shapes and bolts shall be of structural steel.

Chemical and Physical Properties.—The steel when tested shall show the following properties:

TABLE OF CHEMICAL AND PHYSICAL REQUIREMENTS.

	Flange Steel.	Structural Steel.	Rivet Steel.
Maximum Phos., Basic Steel	.04 per cent.	.04 per cent.	.04 per cent.
Maximum Phos., Acid Steel	.08 per cent.	.08 per cent.	.04 per cent.
Maximum Sulphur	.04 per cent.	.05 per cent.	.03 per cent.
Ultimate Tensile Strength, lbs. per Sq. In.	Desired 56,000	Desired 62,000	Not over 55,000 .
Yield point to be observed and recorded in all tests made			
Elongation (in 8 inch and in 2 inch)	26 per cent.	25 per cent.	28 per cent.
Cold Bending	180° flat	180° flat	180° flat
Quench Bending	180° flat		
Nicked Bending Test.			

Stay Rods in Bins.—The stay rods in the regular 14 ft. square bins shall be 1″ diameter upset to 1¼″, and in the divided bins ⅝″ diameter upset to ⅞″. They shall be threaded at ends, as indicated on drawings, with full U. S. standard thread. The rods shall be straight, free from flaws, and shall be annealed after being upset.

Painting.—In shop riveted work the surfaces coming in contact shall be painted one good coat before being put together.

All iron and steel work before leaving the shop shall be thoroughly cleaned, and shall be given one good coat of paint, well worked into all joints and open spaces.

Surfaces which are not accessible for painting after assembling shall have two coats of paint before being assembled. The surfaces in contact of all pieces put together in the field shall each be painted one good coat before being put together.

The paint shall be of a good quality of oxide of iron paint, and shall be approved by the engineers. No painting shall be done in wet or freezing weather, and care shall be taken to have all surfaces dry before treatment.

After erection all of the steel work shall be thoroughly cleaned and all surfaces except the interior surfaces that come in contact with grain (interior surfaces of bins, garners, and hoppers) shall be painted with one coat of the iron oxide paint. After this coat has thoroughly dried all of the above mentioned paint surfaces shall be given a second coat of paint. This coat shall consist of white lead mixed with pure boiled linseed oil colored as may be directed by the engineers. The quality of said paint shall be such as to meet with the approval of the engineers.

All paint shall be thoroughly and evenly applied and well worked into all joints and open spaces. Finished surfaces shall be given one heavy coat of white lead and tallow before being shipped from the shop.

Inspection.—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable machine for testing full-sized members, if required.

When an inspector is furnished by the company he shall have full access at all times to all parts of the shop where material under his inspection is being manufactured.

The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the company.

Material shall not be rolled nor any work done before the company has been notified and arrangements have been made for the inspection.

The engineer shall be furnished complete shop plans and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship. The inspector shall be paid by the company.

Clips.—Steel clips of an approved make shall be used to connect the T's supporting the book tile to the girders.

Erection.—The center of each base must be made true to the column center, as given on the plans, within $\frac{1}{16}$ of an inch, and its height shall be adjusted exactly, using an engineer's level and referring to a fixed bench mark. The anchor bolts for the cast bases will be set in place by the company.

Each cast base shall be set in exact position both as to alignment and to height, using cement grout for this purpose. The casting shall be supported on wooden wedges until the grout has set. Each base shall be thoroughly bedded in the grout. Cement grout for this purpose shall be made with Portland cement and sand. The Portland cement for the work will be furnished by the company. An equal quantity of cement and sand shall be used, and the two thoroughly mixed dry in a tight box. Enough water shall then be added to make the whole just flow under its own weight. The whole operation of mixing and setting shall be performed as rapidly as possible. No grout shall be used that has begun to set. After the bases are set, their heights will be inspected by the engineers, and if they are found to vary more than $\frac{1}{32}$ of an inch from the correct height, they shall be taken up and re-set.

The use of iron sledges in driving or hammering beams or columns or other structural steel will not be allowed where it can be avoided. Wooden mauls shall be used wherever their use is possible. Care shall also be exercised to prevent the materials from falling or from being in any way subjected to heavy shocks.

Especial care shall be used to keep the columns and bin walls plumb and in proper line during erection, and they shall be plumbed to the satisfaction of the engineers as often as they may desire. In case the columns or bin walls are not kept plumb, the entire work of erection shall stop, upon receipt of a written order from the engineers to that effect, and the measures to be employed to remedy the defect shall be approved by the engineers before the erection proceeds.

The use of steel wedges or shims to plumb up columns will not be permitted, the intention being to have the work so accurately done that such practice will be unnecessary.

The sections of columns, truss members, beams or girders must nowhere be cut without first obtaining the approval of the engineers.

Every failure of the material to come together properly shall be noted and reported daily to the engineers. If any serious difficulty occurs during erection, it shall be reported to the engineers before any unexpected measures are used to meet the difficulty.

The contractors shall furnish and erect temporary timber bracing whenever necessary during the erection of the building and whenever so directed by the engineers.

Bins.—The bins shall be made grain tight; any openings in the bin walls, at points where the splices are made, or at any other place in the walls, or bottoms, shall be securely stopped up in a workmanlike manner. After a complete section of bin wall has been erected, the spaces between the column shapes shall be filled with cement grout made in the manner specified for the grout for the column bases, except that it shall be composed of one part Portland cement to four of sand. Care must be taken to stop openings at bottom of the column shapes of lower section until the grout has set.

After each complete section is riveted up and before the grout is poured into the openings between the column shapes, the stay bolts shall be adjusted so as to true up the bin wall. After this has been done, the grout shall be applied in the aforesaid manner.

The work of erection shall be done in a workmanlike manner and to the satisfaction of the engineers.

CONCRETE BINS.—Circular reinforced concrete bins have horizontal and vertical reinforcement. The horizontal reinforcement is either single, when it is placed at the center of the walls, as in the Santa Fe Elevator, Fig. 228; or double, when bars are placed near the surface, as in the Canadian Pacific Ry. Elevator, Port Arthur, Ontario, Fig. 230. The horizontal reinforcement may be continuous, rising from the bottom to the top as a spiral, as in the Santa Fe Elevator, or may be in separate rings. When spiral reinforcement is used the reinforcement is usually high steel wire. The vertical rods are equally spaced and are wired or clamped to the horizontal rods at intersections, see Fig. 228.

Design of Horizontal Reinforcement.—The horizontal reinforcement is usually designed to take all the tensile stresses due to the pressure of the grain. Now let $g =$ vertical spacing of the bars in inches, $p =$ grain pressure per square inch on a point midway between the bars, $d =$ the diameter of the bin in inches, $A =$ net area of steel rods in a height g, and $f =$ allowable unit stress in the rods.

Then the net area of the steel for spacing g will be given by formula

$$A = \frac{p.g.d}{2f} \qquad \text{(a)}$$

and the spacing for rods with an area A will be

$$g = \frac{2A.f}{p.d} \qquad \text{(b)}$$

Design of Vertical Reinforcement.—The vertical reinforcement carries the load between the horizontal reinforcement and takes its proportion of the vertical load. The walls will have negative bending moments at the horizontal reinforcement, and positive bending moments midway between the horizontal reinforcement. The bending moments on a unit width are approximately

$$M = \pm \tfrac{1}{12} p.g^2 \qquad \text{(c)}$$

Then the area of the steel can be calculated by formula (56).

The pressure on any horizontal section is equal to the weight of the wall plus the weight of the grain carried by the walls. This pressure is carried by the concrete and the steel, the stress in the steel being n times the stress in the concrete, where $n = E_s \div E_c$.

Wind Stresses.—The stresses due to wind moment in grain bins are small and the concrete and steel may both be assumed to take tension and compression. The wind pressure will act at one-half the height and will produce a moment M, about the center of any horizontal section.

Then the extreme fiber stress in the concrete will be given by the formula

$$c = \pm \frac{M.d}{2(I_c + n.I_s)} \qquad \text{(d)}$$

Where d is the external diameter of the bin.

The stresses in the steel will be given by the formula

$$f = \frac{n.M.d'}{2(I_c + n.I_s)} \qquad \text{(e)}$$

Where d' is the diameter of the reinforcement. With vertical reinforcement of ½″ square bars spaced 12″ to 18″ in grain bins of ordinary sizes the total stresses are nominal.

Example 1.—As an example the reinforcement in the reinforced concrete grain bin in Fig. 228 will be investigated. A section 70 feet below the top will be taken. The diameter = 24′ 2″; the spacing $g = 7''$ and $f = 16{,}000$ lbs. per sq. in., and $p = 4.3$ lbs. per sq. in. (Fig. 153).

Horizontal Reinforcement.—Then

$$A = \frac{4.3 \times 7 \times 24.16 \times 12}{2 \times 16{,}000} = 0.27 \text{ sq. in.}$$

The rods are ¾″ round with an area of 0.44 sq. in., which makes a stress of 10,000 lbs. per sq. in. in the gross section of the horizontal rods. If the splice has an efficiency of 0.75, the stress in net area will be about 13,400 lbs. per sq. in.

Vertical Reinforcement.—The approximate weight of the concrete above this horizontal joint, = 396,000 lbs., = 5,000 lbs. per lineal foot. The weight of the grain carried by the walls is from Fig. 157, = 859,000 lbs., = 11,900 lbs. per lineal foot. The vertical bars are ½″ □ and are spaced 18″ centers. The area of steel per lineal foot is then, $A = 0.167$ sq. in. The area of concrete = 84 sq. in. The concrete is 1 : 2½ : 4 and $n = 10$.

Then

$$c = \frac{5{,}000 + 11{,}900}{84 + 10 \times 0.167}$$

$$c = 195 \text{ lbs. per sq. in.}$$

and

$$f = 1{,}950 \text{ lbs. per sq. in.}$$

The wind pressure will be taken at 20 lbs. per sq. ft. of vertical projection, and $M = 20 \times 70 \times 24.16 \times 35 \times 12 = 14{,}206{,}080$ in.-lbs.

Now

$$I_c = \frac{\pi}{64}[(24.16 \times 12)^4 - (23 \times 12)^4] = 61{,}957{,}600 \text{ in.}^4$$

The value of I_s will be very nearly $\frac{1}{36}$ of I_s for a $\frac{1}{2}''$ ring of steel.

$$I_s = \frac{\pi}{64} \times \tfrac{1}{36}[(283\tfrac{1}{2})^4 - (282\tfrac{1}{2})^4] = 123{,}520 \text{ in.}^4$$

Now from (d)

$$c = \pm \frac{M.d}{2(I_c + n.I_s)} = \pm 35 \text{ lbs.}$$

and

$$f = \pm 350 \text{ lbs.}$$

The stress in the vertical reinforcement as given by formula (c) is nominal. The maximum compression in the concrete is then $c = 195 + 35 = 230$ lbs. per sq. in. This gives a factor of safety of about 10 in the concrete. The vertical reinforcement is ample.

Columns.—Concrete columns are reinforced (a) with longitudinal reinforcement, (b) with spiral or hooped reinforcement; or (c) with both longitudinal and hooped reinforcement.

Experiments have shown that for concrete columns reinforced with longitudinal reinforcement and having a length not to exceed 25 times the least diameter or dimension the ultimate strength will be approximately given by the formula

$$P = 2{,}000\,(A_c + n.A_s) \qquad \text{(f)}$$

where P = ultimate strength in lbs., A_c = area of concrete, A_s = area of steel, both in sq. in., and $n = E_s \div E_c$. For safe loads take a factor of safety of 4 to 6.

Experiments by Considere and others show that the value of metal in hooping is from 2 to 2.4 times the value of metal in longitudinal reinforcing bars. While the metal in the longitudinal bars has less efficiency it should not be omitted for the reason that columns reinforced longitudinally and spirally are more reliable than columns reinforced in a single way. The pitch of the spirals should be kept below $\frac{1}{8}$ to $\frac{1}{7}$ of the diameter of the column.

For the different methods of reinforcing beams and columns, and for details of construction, see Buel and Hill's "Reinforced Concrete," and other standard works on reinforced concrete.

SPECIFICATIONS FOR CONCRETE STORAGE BINS.—Complete specifications for the concrete storage bins of the Santa Fe Grain Elevator, Chicago, Ill., are given in Chapter XIX. These specifications were prepared by the John S. Metcalf Co., Chicago, Ill.

The following description and abstract of the specifications for the concrete storage bins of the Missouri Pacific Ry. Co. at Kansas City, Mo. (see Fig. 148), will give a clear idea of the details of construction. These specifications were prepared by the above company.

Concrete Storage Bins.—The storage bins are 42′ 0″ outside diameter and 81′ 6″ high, with walls of concrete 9″ thick, reinforced with steel rods. The concrete is composed of one part Portland cement, $2\frac{1}{2}$ parts sand, and 4 parts broken stone that will pass a 1″ ring, or such proportions of cement, sand, and stone as will completely fill the voids. The horizontal reinforcing rods are $\frac{3}{4}$″ in diameter; they are spaced 3″ apart for a distance of 11′ 6″ from the bottom of the bin; $3\frac{1}{2}$″ apart for the next 12′ 3″; 4″ apart for the next 12′ 0″; 5″ apart for the next 8′ 9″; 6″ apart for the next 9′ 0″; 8″ apart for the next 6′ 0″; 10″ apart for the next 5′ 0″; and 12″ apart for the remaining 17′ 0″. These rods are bent to the curve of the bin, and have their ends bent and connected by links with wedges as shown in Fig. 228. The ends of each band are lapped at least 18″. The several bands of horizontal rods are erected so as to break joints. The vertical rods are $\frac{5}{8}$″ in diameter and are spaced 18″ centers, or 86 in each bin. They are composed of sections not less than 6 feet long, connected by cast iron couplings, and the adjacent rods are arranged to break joints with each other. The lapped portions of the horizontal rods and the crossings of the horizontal and vertical rods are to be properly fastened together with No. 14 annealed wire, or with steel clips.

The forms are to be so constructed and manipulated that the walls of the bins shall be perpendicular, and free from ledges or pockets on which the grain can lodge. The concrete shall be deposited in a state sufficiently fluid to form walls that are smooth and free from voids. After the molds are removed all pits or voids are to be carefully pointed up with one to one Portland cement mortar. No plastering of any surface will be allowed. The bottoms of the bins are to be made of concrete of the same proportions as for the bin walls. The surface of the bottoms are to be given a trowel finish. The molds and the method of erection are shown in Fig. 200.

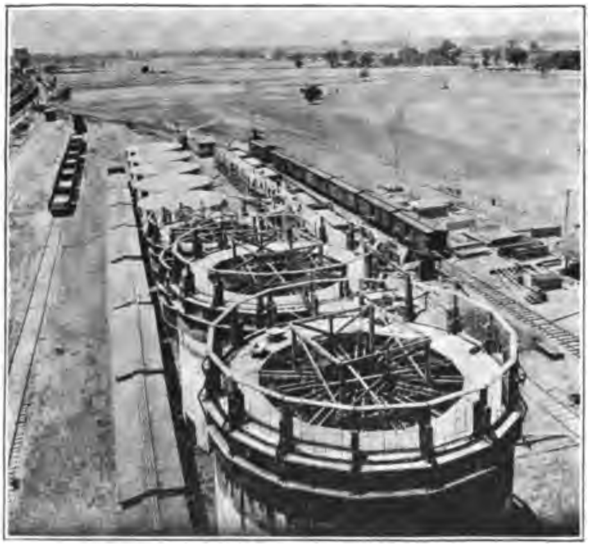

FIG. 200. PROGRESS VIEW OF MISSOURI PACIFIC RY. CONCRETE GRAIN BINS, SHOWING FORMS FOR CONCRETE.

The roofs of the tank are to be made of 3″ book tile supported on steel purlins, covered with a tar and gravel roofing. The floor of the conveyor gallery is to be made of 3″ book tile, supported on a steel framework, and covered with a top dressing composed of one part Portland cement and two parts sand. Each cylindrical bin shall be provided with a bin ladder, and one fire escape shall be provided as shown.

The interspace bins are frequently used for grain storage. Where these bins are used they should be braced with rods and the intersections of the circular bins should be thickened and strengthened. Several failures due to faulty construction have occurred in concrete bins where the interspaces have been used for storage.

SPECIFICATIONS FOR TILE BINS.*—"The tank walls will be constructed of special semi-porous hollow tile made to conform to the circle of the tank. The main wall tile shall be 12" × 12" × 6", the channel tile 12" × 4" × 5", the outside facing tile made of similar material, but semi-glazed are 12" × 12" × 1¼". All the tile shall be well manufactured, hard burned and reasonably free from cracks and defects. The wall tile shall be set in the wall with the cells running vertically and all set with full flush mortar joints sides and bottoms. The joints shall be thoroughly pointed on both sides of the wall and the top of each course of wall tile shall be covered with a metal wire fabric, about four meshes to the inch, so as to give a full mortar joint for the next course of channel tile. The channel tile shall be set to

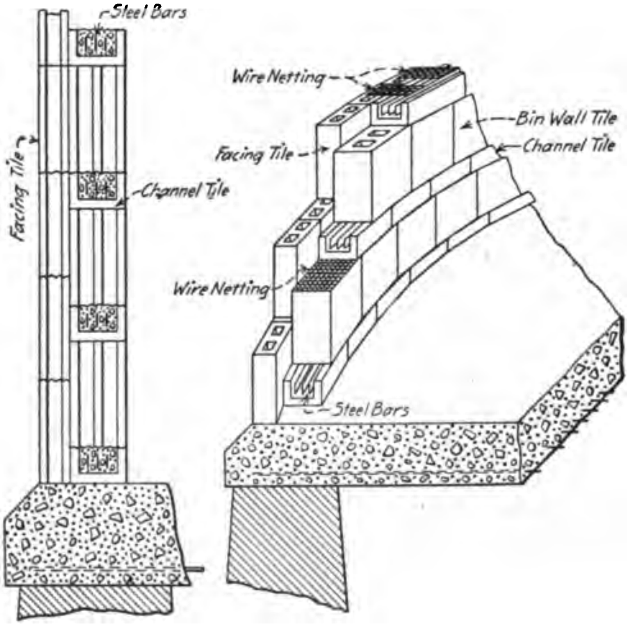

FIG. 201. DETAILS OF BARNETT-RECORD TILE GRAIN BIN.

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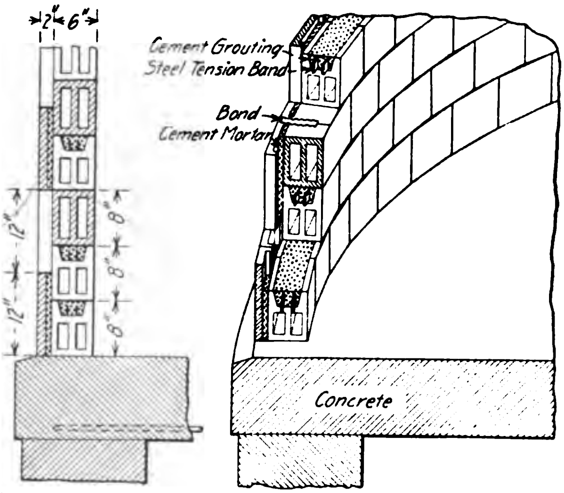
* Prepared by the Barnett-Record Co.

break joints with the wall tile and, after a full course has been set, steel bands of the sizes shown on the drawings shall be placed in same and carefully spaced so as to be separate from each other and from the sides of the tile. Each course of channel tile shall contain at least two complete steel bands. When the bands have been properly placed in the channel tile the same shall be filled with thin Portland cement mortar, composed of one part sand and two parts Portland cement, and thoroughly worked around the bands to insure that all the bands are thoroughly coated and that all the crevices are filled. The facing tile are laid in courses with a full mortar bed and are tied to the main wall of tanks by wire fabric, similar to that used for the main wall tile."

The method of laying the tile and the general appearance of the bins is shown in Fig. 151 and Fig. 201.



Section through Bin Wall.

FIG. 202. DETAILS OF WITHERSPOON-ENGLAR TILE GRAIN BIN.

For a complete description of a tile storage elevator, see description of the Canadian Pacific grain elevator, Port Arthur, Ontario, in Chapter XIX.

The details of the Witherspoon-Englar tile bin are shown in Fig. 202. This system appears to be somewhat more simple than the system shown in Fig. 201.

BIN ROOFS. *Steel Bins.*—The most satisfactory roof for steel bins is a conical steel sheet roof supported on a radial framework.

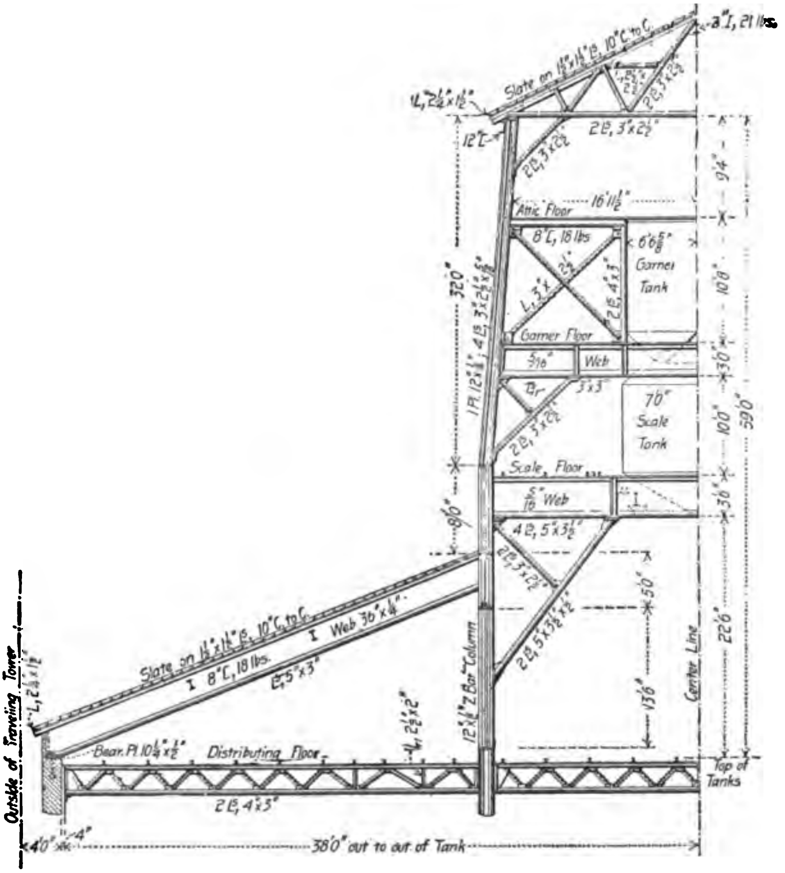


FIG. 203. STEEL ROOF FRAMING OF GREAT NORTHERN ELEVATOR, BUFFALO, N. Y.

For this roof No. 14 sheet steel is about the lightest metal that can be calked. The sheets should be cut radially and should be riveted with light rivets spaced 2½ to 3 inches; the rivets are usually driven cold.

The roof sheets are sometimes detailed developed and the sheets are bent to shape. In this case the sheets are rectangular. It is very difficult to make a presentable job in this way. The radial framework for the steel bins of the Independent Elevator at Omaha, Neb., is shown in Fig. 212, the detail plans of the steel roof are shown in Fig. 213, and the details of a radial steel truss are shown in Fig. 214. Steel roofs sometimes sweat on the inside. This may be prevented by the use of an "anti-condensation" lining as described in "The Design of Steel Mill Buildings."

The steel framing and roof above the bins of the Great Northern elevator, Buffalo, N. Y., is shown in Fig. 203.

Tile Roofs.—The roofs of tile and concrete bins are usually made of book tile supported on sub-purlins carried on a radial framework, as shown in Fig. 212. The book tile are then covered with a composition tar and gravel roof covering.

See detailed descriptions of the tile roof of the reinforced concrete storage bins for the Missouri Pacific Ry., given on a preceding page. For additional details see Chapter XIX.

Cement Roofs.—A reinforced concrete roof supported by the radial framework is often used on concrete, tile or brick bins. The concrete is usually covered with a tar and gravel roof covering.

FOUNDATIONS.—Grain bins should be built on solid foundations, or on monolithic foundations if the soil is yielding. Bin failures have occurred in bins with inadequate foundations by the outside ring being forced down by the load of the grain on the bin walls, and the center being forced up by the upward pressure of the soil.

Bin bottoms are commonly made of concrete, reinforced or plain, carried on a sand or gravel filling placed inside the bin. The conveyor tunnels pass under the bins, the grain discharging from the bin through one or more spouts onto the conveyor belts. The bins should discharge from as near the center as practicable. Care should be used to waterproof the floor of the bins or the grain will absorb moisture and spoil. The floor should be drained with drain tile set just below the finish or a layer of asphalt or other waterproofing should be used in the concrete layer.

Small storage bins are sometimes made with flat bottoms, in which case part of the grain must be shoveled out. Plate bottoms are sometimes used for small bins, and the grain is sometimes discharged from the side. Eccentric discharge gates cause excessive lateral stresses when the bin is emptying (see Chapter XVII).

For details of bin foundations see Figs. 211, 228, 231, 232, 236, and 238.

CONVEYORS.—Grain is commonly moved in a horizontal direction in large elevators by means of belt conveyors, while in small elevators the screw conveyor is most in use for this purpose. Movement in a vertical direction is by belt or chain conveyors in closed boxes called elevator legs. For details of conveyors see Chapter XIV.

Stresses in a Steel Bin Due to Wind Moment.—If M is the moment due to the wind acting on the bin above the horizontal joint, then the stress per lineal inch of joint due to wind moment will be

$$S = \frac{M.d}{2I}, \qquad \text{but } I = \tfrac{1}{8}\pi.d^3 \text{ (approx.)}$$

and

$$S = \frac{4M}{\pi.d^2} \tag{a}$$

In the example on page 286 by substituting in (a) we have, $S =$ 300 lbs. per lineal inch, or 3,600 lbs. per lineal foot. The space should therefore not be greater than $3,240 \times 12 \div 16,100 = 2\tfrac{3}{8}''$.

CHAPTER XIX.

EXAMPLES OF GRAIN ELEVATORS.

Introduction.—In this chapter complete descriptions are given of the Independent Steel Elevator, Omaha, Neb.; the Great Northern Ry. steel elevator, West Superior, Wis.; the Santa Fe Ry. concrete elevator, Chicago, Ill.; the Canadian Pacific Ry. Co.'s concrete grain elevator, Port Arthur, Ontario; and the Canadian Northern Ry. Co.'s tile grain bins, Port Arthur, Ontario.

The author is under obligations to the Minneapolis Steel & Machinery Co., Minneapolis, Minn., for plans and specifications for the Independent Steel Elevator; also to the John S. Metcalf Co., Chicago, for plans and specifications of the Santa Fe Ry. elevator.

THE INDEPENDENT STEEL ELEVATOR, OMAHA, NEB.

General Description.—This elevator consists of a steel working house having a bin capacity of 240,000 bushels and 8 steel storage bins having a storage capacity of 100,000 bushels each, making a total storage capacity of 1,040,000 bushels. A view of the completed structure is shown in Fig. 204, and a progress view is shown in Fig. 205.

The steel working house is 64′ 0″ × 70′ 0″, with 14′ 0″ sheds on two ends and one side, as shown in Fig. 206. The sub-story of the building is 26′ 0″. The bins are 65′ 4″ high, as shown in Fig. 209, and are supported on steel columns, as shown in Fig. 207 and Fig. 208. The spouting story is 24′ 6″ high; the garner and scale story is 26′ 6″ high; and the machinery story is 13′ 8″ high. The walls below and above the bins are covered with No. 24 corrugated steel laid with 1½ corrugations side lap and 3 inches end lap. The roof is covered with No. 22 corrugated steel laid directly on the steel purlins with 2 corrugations side lap and 6 inches end lap.

On the first or working floor the floor between the tracks is made

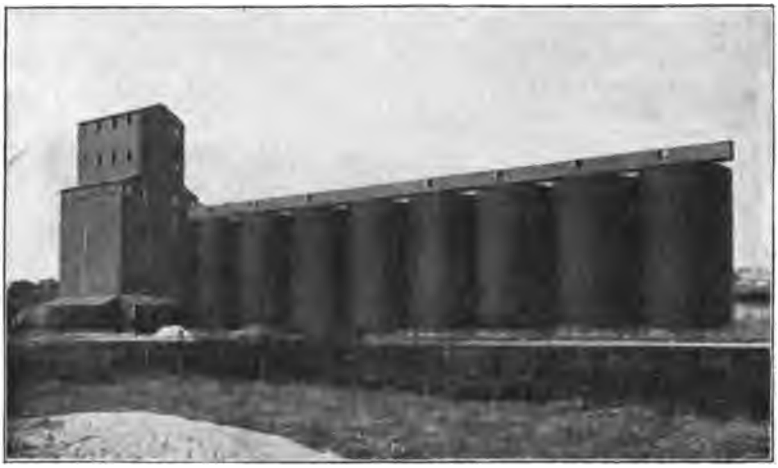


FIG. 204. THE INDEPENDENT ELEVATOR, OMAHA, NEB. 1,040,000 BUSHEL CAPACITY.

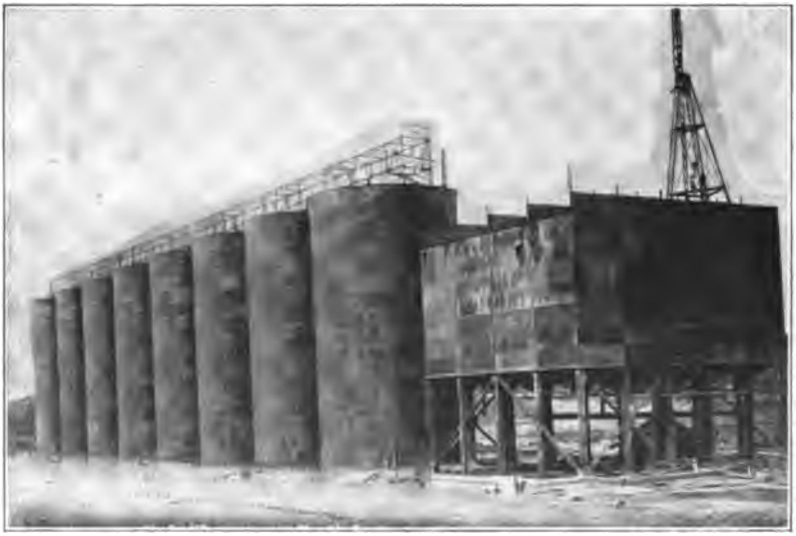


FIG. 205. THE INDEPENDENT ELEVATOR UNDER CONSTRUCTION.

of ¼ inch plate bolted to the beams, while the remainder of this floor is made of concrete filled in above concrete arches which rest on the flanges of the beams with a finish 1½″ thick of Portland cement mortar consisting of one part cement to one part clean, sharp sand. The concrete is composed of one part Portland cement, two parts sand, and five parts crushed stone.

The floor of the cupola throughout the different floors and in the gallery leading over the bins is made of No. 24 corrugated steel resting on steel framework, and covered with 3 inches of concrete and a one-inch finish of one to one Portland cement mortar troweled smooth. All doors are of the rolling steel type. The window frames were made of 2″ × 6″ timbers and are covered with No. 26 sheet steel. All windows are provided with 1¾″ checked rail sash and are glazed with double strength glass.

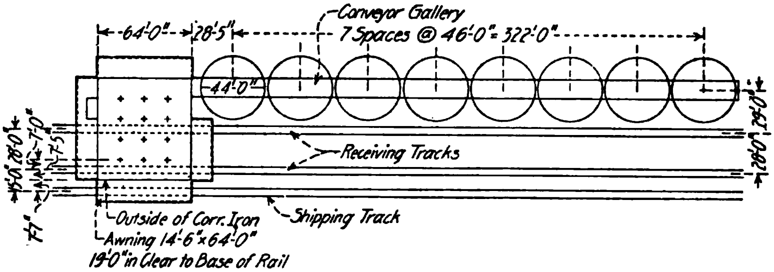


FIG. 206. PLAN OF INDEPENDENT ELEVATOR.

Painting.—All steel work of every description was painted with one coat oxide of iron paint at the shop and a second coat after erection. The tank plates and corrugated steel were painted on the exterior surface only after erection.

Bins.—The eight steel storage bins are 44′ 0″ in diameter and 80′ 0″ high, have a capacity of 100,000 bushels and rest on separate concrete foundations. The bins are constructed of steel plates stiffened with Z-bars, as shown in Fig. 210. The bins are covered with a steel plate roof, Fig. 213, supported on roof trusses, as shown in Fig. 212 and Fig. 214. A conveyor gallery 10′ 0″ wide and 8′ 0″ high

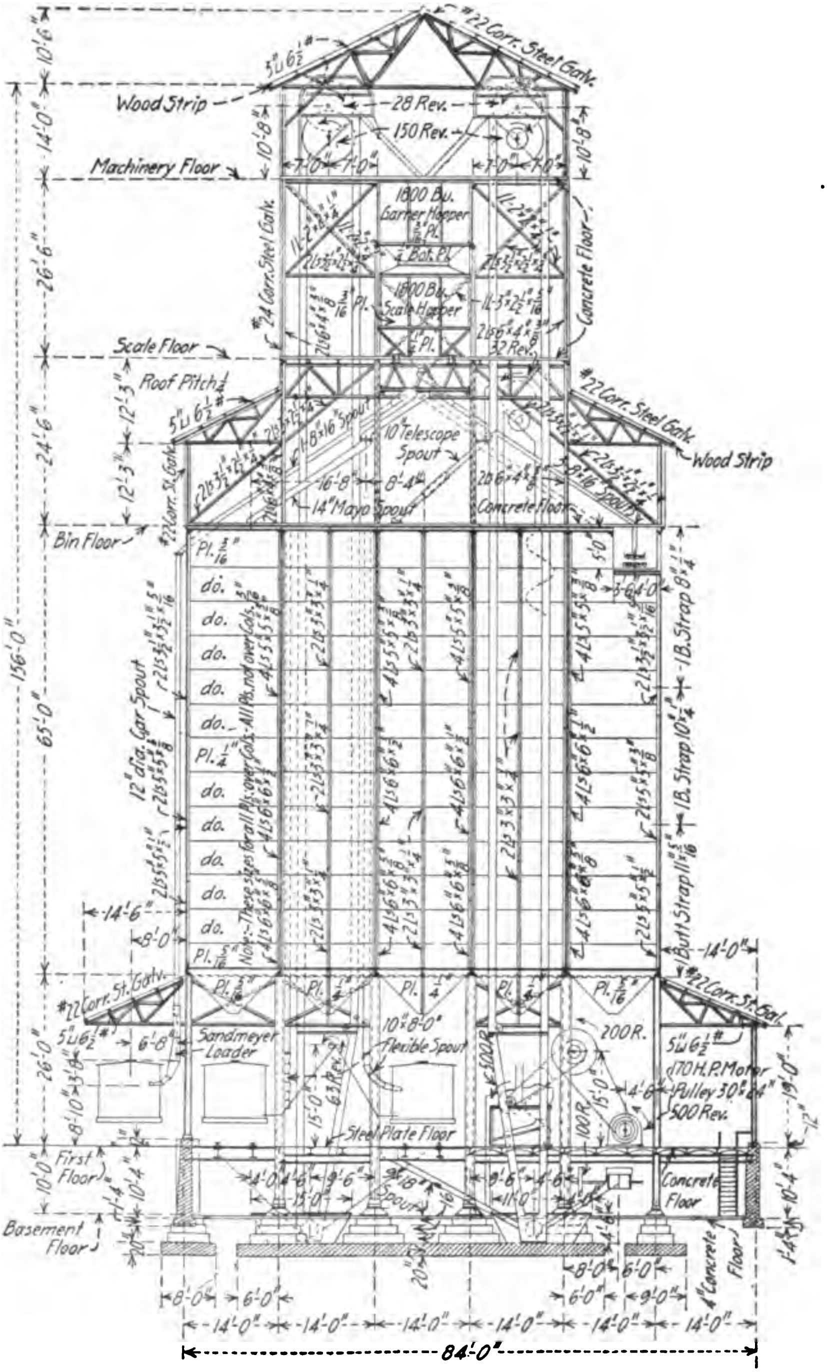
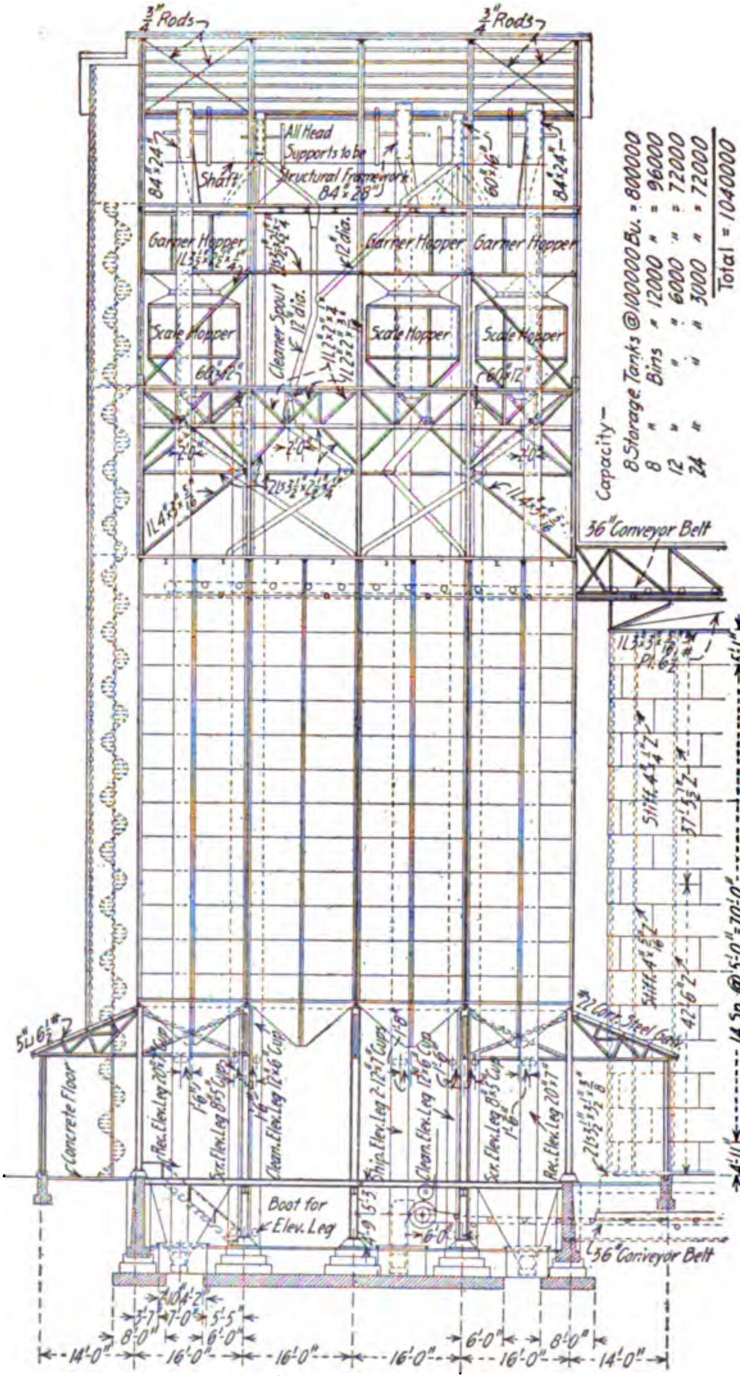


FIG. 207. TRANSVERSE SECTION OF WORKING HOUSE OF INDEPENDENT ELEVATOR.



Longitudinal Section.

FIG. 208. LONGITUDINAL SECTION OF WORKING HOUSE OF INDEPENDENT ELEVATOR.

extends from the working house over the bins. A conveyor tunnel extends from the working house under the bins. The rivet spacing in the circular bins is shown in Fig. 211.

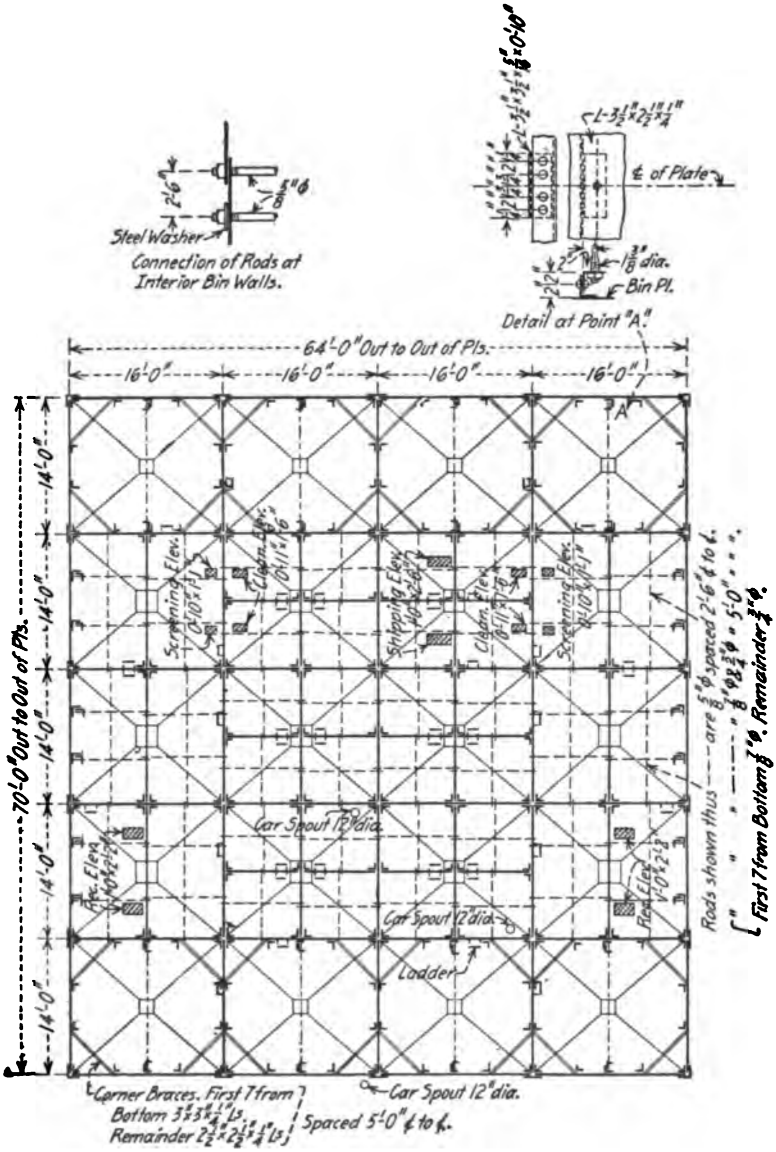

Steel Washer
Connection of Rods at Interior Bin Walls.
2'-6"
5/8"φ
L-3½"×2½"×¼"
℄ of Plate
1⅜" dia.
Bin Pl.
Detail at Point "A."
64'-0" Out to Out of Pls.
16'-0"
16'-0"
16'-0"
16'-0"
70'-0" Out to Out of Pls.
14'-0"
14'-0"
14'-0"
14'-0"
14'-0"
A
Screening Elev.
Clean. Elev.
Shipping Elev.
Clean. Elev.
Screening Elev.
Rec. Elev.
Rec. Elev.
Car Spout 12" dia.
Car Spout 12" dia.
Ladder
Rods shown thus ——— are 5/8"φ spaced 2'-6" ℄ to ℄.
" " —·—·— " 7/8"φ & ¾"φ " 5'-0" " " ".
First 7 from Bottom 7/8"φ. Remainder ¾"φ.
Corner Braces. First 7 from Bottom 3"×3"×¼" Ls. Remainder 2½"×2½"×¼" Ls. Spaced 5'-0" ℄ to ℄.
Car Spout 12" dia.

FIG. 209. PLAN OF BINS IN WORKING HOUSE OF INDEPENDENT ELEVATOR.

The bins in the working house are arranged as shown in Fig. 209, and are constructed of plates, as shown in Fig. 207 and Fig. 208. The bins, 14′ 0″ × 16′ 0″, are braced in the corners with angle braces spaced 5′ 0″ centers vertically, and of the sizes shown in Fig. 209. The large bins are also braced with $\frac{7}{8}$ and $\frac{3}{4}$-inch round rods spaced 5′ 0″ apart as shown. All the smaller bins are braced with $\frac{5}{8}$-inch round rods spaced 2′ 6″ apart as shown. Vertical angles in the sides of the bins are provided, as shown in Fig. 207, Fig. 208, and Fig. 209.

EQUIPMENT.—There are two stands of receiving elevators with receiving pits on either side. These elevators have 22-inch 6-ply belts and 20″ × 7″ × 7″ buckets spaced 14 inches apart; the receiving pits are covered with steel grating, and a pair of Clark's automatic grain shovels are located at each unloading place. These elevators are driven with an electric motor of 100 H. P., each elevator being driven with a clutch and pinion so that the elevator may be stopped and started at will.

There is one stand of shipping elevators constructed in the same manner, having a 26-inch 6-ply belt and 2 lines of 12″ × 7″ × 7″ buckets spaced 14 inches apart.

There are two stands of cleaning elevators with 14-inch 6-ply belts with 12″ × 6″ × 6″ buckets spaced 12 inches apart.

There are also two screenings elevators with 9-in 5-ply belts with 8″ × 5″ × 5″ buckets spaced 12 inches apart.

The shipping, screenings, and cleaner elevators are driven from a line shaft which is driven by a 100 H. P. motor, each elevator being driven by a core wheel and pinion.

Three scale hoppers, having a capacity of 1,800 bushels, are located in the cupola, and three garner hoppers of 1,800 bushels capacity are located above the scale hoppers.

The main line shaft on the first floor is driven by a 170 H. P. motor.

A car puller capable of moving 25 loaded cars is provided.

Elevators.—The boots of the receiving and shipping elevators rest in water-tight steel boot tanks made of $\frac{5}{16}$-inch steel plates. The ele-

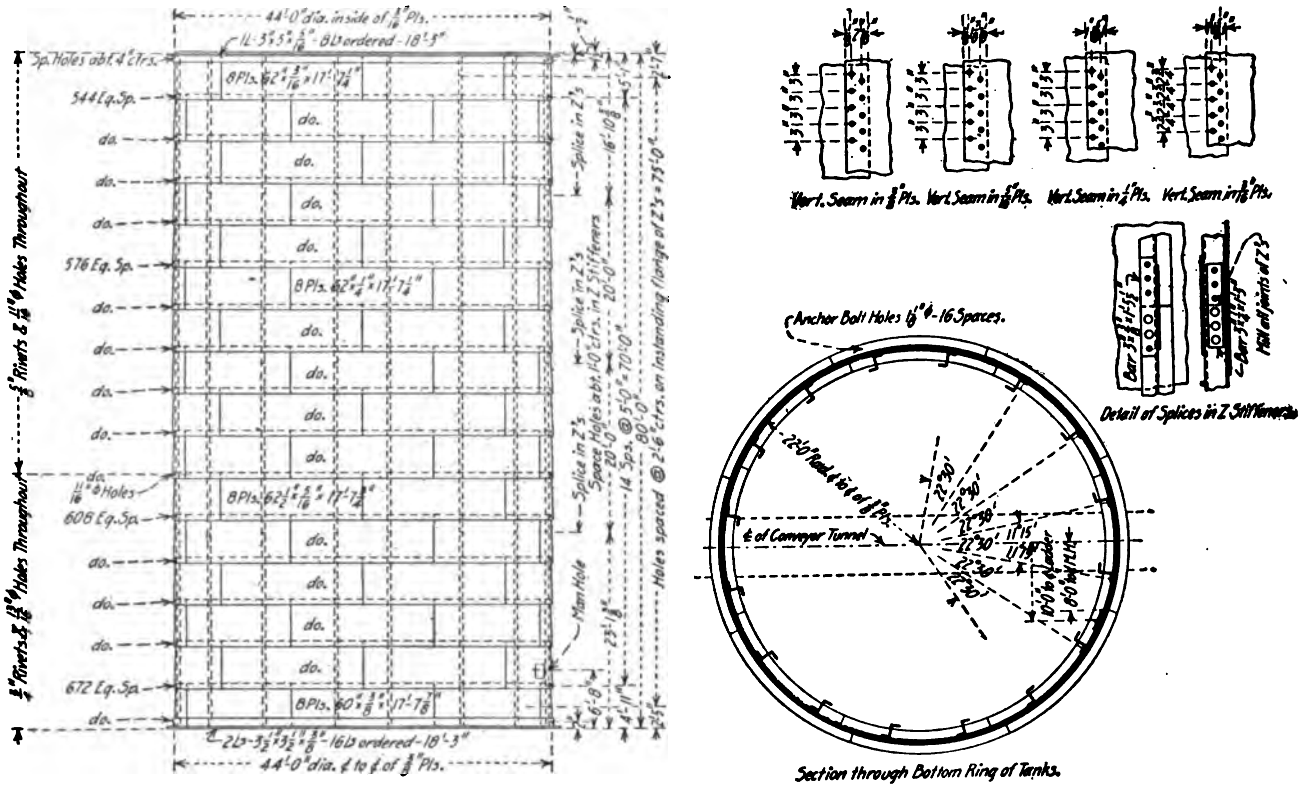

44'-0" dia. inside of 3/16" Pls.
1L-3"×5"×5/16"-8 Ls ordered-18'-3"
Sp. Holes abt. 4" Ctrs.
8 Pls. 62"×3/16"×17'-7 1/4"
544 Eq. Sp.
do.
576 Eq. Sp.
8 Pls. 62"×1/4"×17'-7 1/4"
11/16" Φ Holes
8 Pls. 62 1/2"×5/16"×17'-7 1/4"
608 Eq. Sp.
672 Eq. Sp.
8 Pls. 60"×3/8"×17'-7 7/8"
2Ls-3 1/2"×3 1/2"×3/8"-16Ls ordered-18'-3"
44'-0" dia. ℄ to ℄ of 3/8" Pls.
5/8" Rivets & 11/16" Φ Holes Throughout
3/4" Rivets & 13/16" Φ Holes Throughout
Splice in Z's
Space Holes abt. 1'-0" ctrs. in Z Stiffeners
Man Hole
16'-10 5/8"
20'-0"
23'-1 3/8"
6'-8"
5'-1"
4'-11"
14 Sps. @ 5'-0" = 70'-0"
80'-0"
Holes spaced @ 2'-6" ctrs. on Instanding flange of Z's = 75'-0"
2'-7"
2'-5"
Vert. Seam in 3/8" Pls.
Vert. Seam in 5/16" Pls.
Vert. Seam in 1/4" Pls.
Vert. Seam in 3/16" Pls.
Bar 3 1/2"×1/4"×1'-5 1/2"
Mill all joints of Z's
Detail of Splices in Z Stiffeners
Anchor Bolt Holes 1 1/8" Φ-16 Spaces.
22'-0" Rad. ℄ to ℄ of 3/8" Pls.
℄ of Conveyor Tunnel
22°30'
11°15'
10'-0" to Ladder
6'-0" to M.H.
Section through Bottom Ring of Tanks.

FIG. 210. DETAILS OF STEEL BINS FOR INDEPENDENT ELEVATOR.

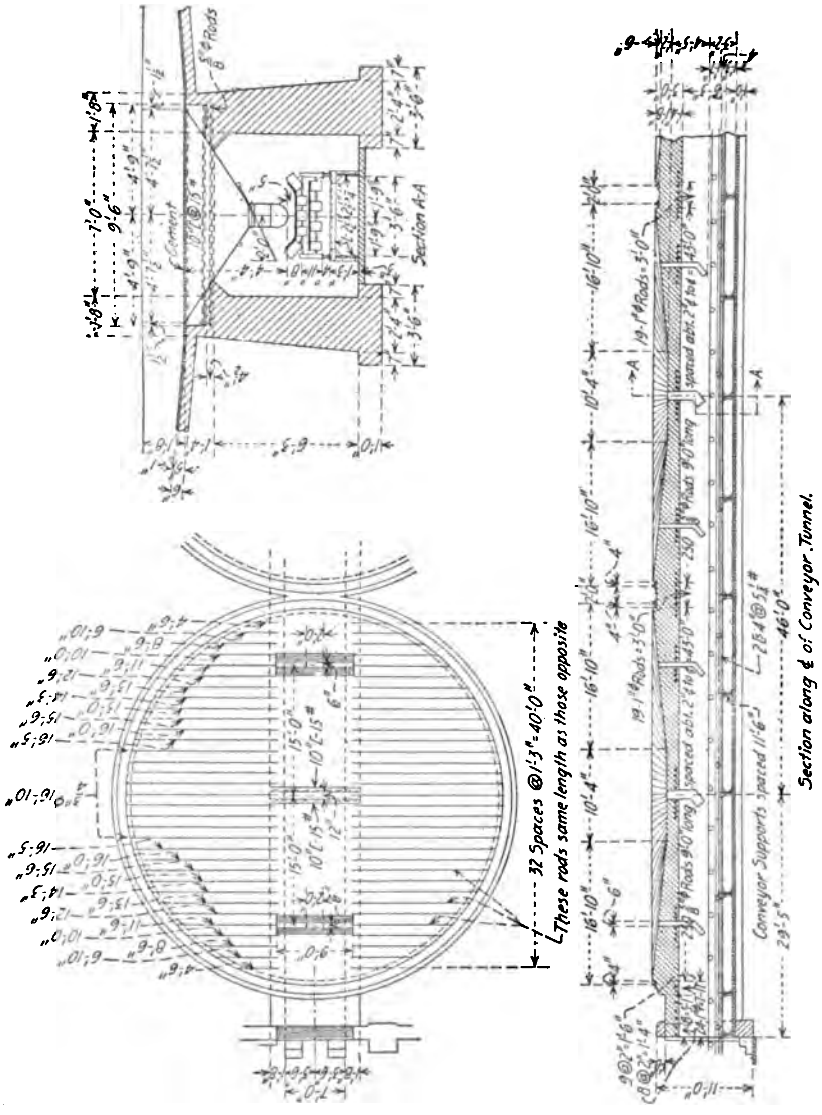

Section A-A
Cement
Section along ℄ of Conveyor Tunnel.
Conveyor Supports spaced 11'-6"
32 Spaces @1'3"=40'0"
These rods same length as those opposite
15'-0"
10"⊏-15#
46'-0"
29'-5"

FIG. 211. DETAILS OF BIN BOTTOMS AND CONVEYORS UNDER BINS, INDEPENDENT ELEVATOR.

vator boots are made of $\frac{3}{16}$-inch steel plates, the boot pulleys having a vertical adjustment of 8 inches. The elevator cases are made of No. 12 steel up to the bins, and of $\frac{3}{16}$-inch plates in the bins, and No. 14 steel above the bins. The cases are strengthened by angles at the corners. The elevator heads are made of No. 14 steel. At each receiving elevator is a large elevator pit extending from the leg back to the center of the track. This pit is constructed of beams and $\frac{3}{16}$-inch plates and is covered with a grating of $1\frac{3}{4} \times \frac{1}{4}$-inch bars spaced $1\frac{1}{4}$ inches apart.

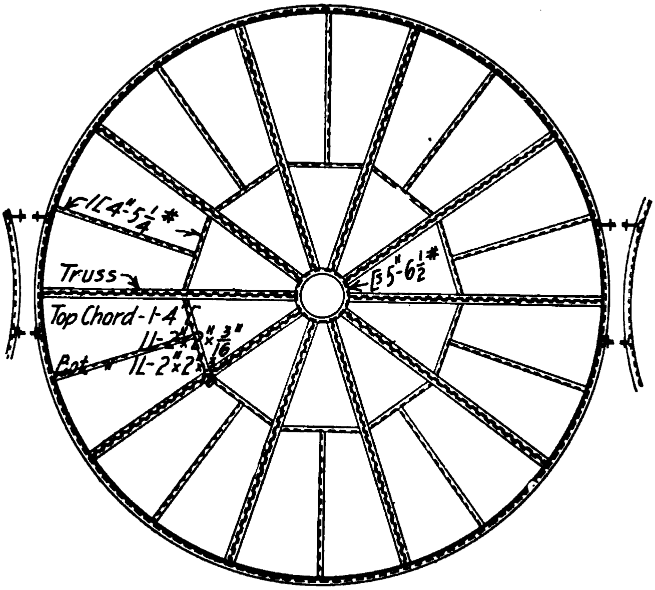
The elevator buckets are "Buffalo" buckets; those for the receiving elevators are 20" × 7" × 7"; for the shipping elevators two lines of 12" × 7" × 7" buckets; for the cleaning elevators one line of 12" × 6" × 6" buckets; and for the screenings elevator one line of 8" × 5" × 5" buckets. The buckets in the receiving, shipping and cleaning elevators are spaced 14" apart, while those in the screenings elevator are spaced 12" apart.

The elevator belts in the receiving elevators are 22 inches wide and 6-ply, the shipping belts are 26 inches wide and 6-ply; the cleaning belts are 14 inches wide and 6-ply, and the screenings belts are 9 inches wide and 5-ply. The belting is made of 32 ounce duck and is first-class.

Spouts.—The building is provided with a complete system of spouts. The general distributing spouts from the scales to the shipping spouts are double-jointed Mayo spouts. There are three shipping spouts which are provided with telescoping bottom sections. All bin bottoms are provided with a revolving spout with a cut-off gate operated with a rack and pinion, with cords leading to within reaching distance of the floor.

Conveyors.—The conveyor belt leading from the working house over the bins is a 36" 4-ply conveyor belt, is carried on disc rolls consisting of 3 straight-faced 6-inch pulleys and 2 special discs; the discs run loose on the shafts, which are $1\frac{3}{16}$-inch diameter and are spaced 5 ft. centers. The return rolls are 5-inch straight-faced rolls spaced 15 ft. centers. At each point in the elevator where grain is loaded onto the belt there are two pairs of special concentrating rolls. Movable trippers provided with spouts are provided, so that grain may be discharged on either side of the belt. The entire conveyor is carried on a steel framework. The conveyor belt is driven by a 40 H. P. motor.

The conveyor in the tunnel leading from the storage tanks to the working house is of the same type as the conveyor above the bins, and is supported on a steel framework, except that the top or carrying rolls are all of the concentrating types, as shown in Fig. 211. The concentrating rollers are composed of two straight-faced rolls from the main shaft, and two concentrating rolls meeting at an angle of 45° to the straight rolls. The lower conveyor is driven by a rope drive from the main line shaft in the working house.



Roof Framing Plan for Tanks.

FIG. 212. FRAMING FOR ROOF OF CIRCULAR BINS, INDEPENDENT ELEVATOR.

Scale Hoppers.—There are three scale hoppers of 1,800 bushels capacity, each mounted on a Fairbanks-Morse and Company's scales, having a capacity of 84,000 lbs., and have steel frames. The hoppers have $\frac{3}{16}$-inch steel plate sides, and $\frac{1}{4}$-inch plate bottoms, stiffened with angle irons, and are tied together with tie rods. Each hopper is provided with a 22-inch cast iron outlet with a steel plate cut-off gate.

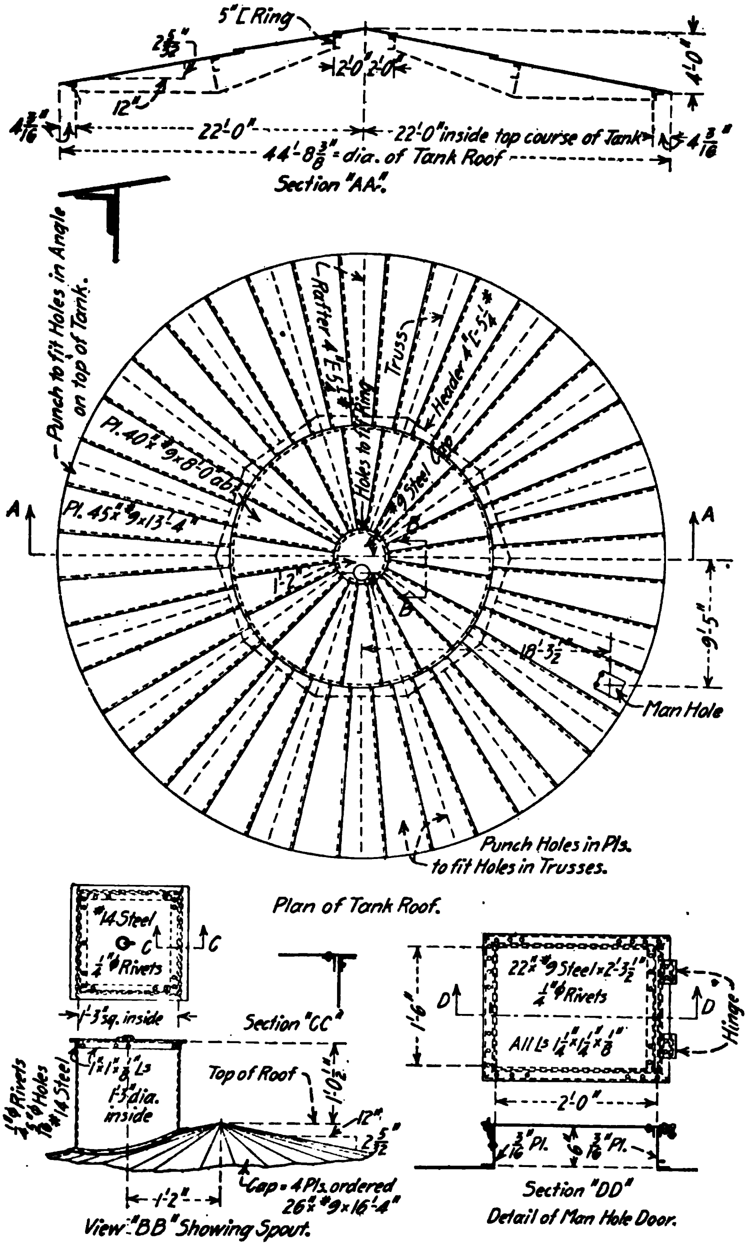


FIG. 213. DETAILS OF STEEL ROOF FOR STEEL BINS FOR INDEPENDENT ELEVATOR.

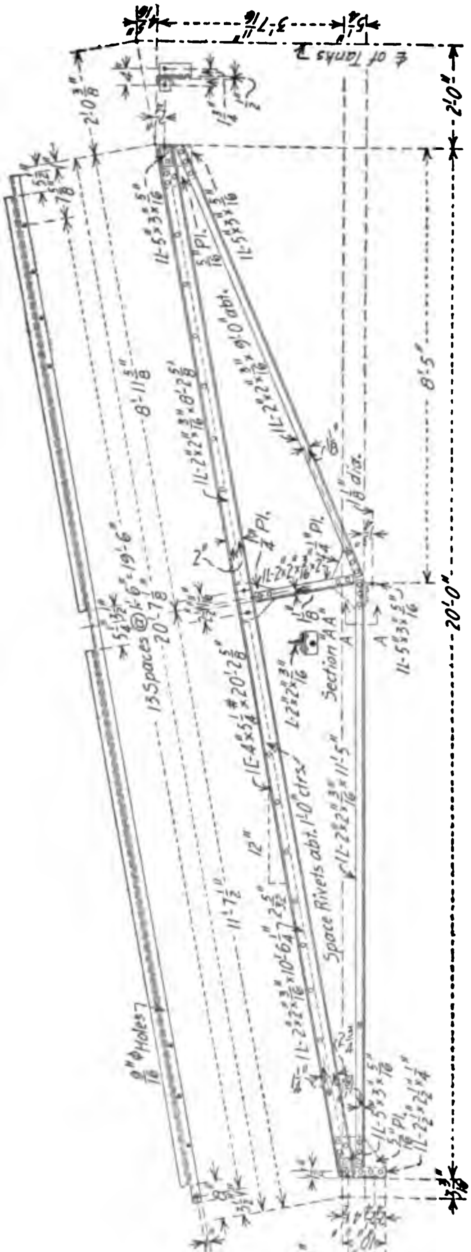


FIG. 214. DETAILS OF STEEL ROOF TRUSS FOR STEEL BINS, INDEPENDENT ELEVATOR.

Garners.—A steel garner hopper is placed directly over each scale hopper. The garners have a capacity of 1,800 bushels, and are constructed with $\frac{3}{16}$-inch side plates and $\frac{1}{4}$-inch bottom plates. The bottoms of the garners are hoppered to four openings, which are provided with gates sliding on steel rollers.

Cleaning Machines.—A large size cleaning machine and a large size oat clipper are provided. These machines are connected with a large dust collector which discharges the dust from the cleaning machines and from the sweepings outside of the building.

Car Puller.—A car puller having a capacity of 25 loaded cars is provided. The car puller has two drums, each provided with 400 feet of $\frac{5}{8}$-inch crucible steel cable.

Shovels.—A pair of Clark automatic grain shovels, with all necessary counterweights, sheaves, scoops, etc., are provided.

The total weight of steel in the elevator is 1,700 tons; approximately 900 tons in the working house, and 800 tons in the circular bins and conveyors.

The total cost was $205,000, of which the 8 steel bins and conveyors cost $80,000.

STEEL GRAIN ELEVATOR, GREAT NORTHERN RY., WEST SUPERIOR, WIS.*

General Description.—The elevator is rectangular in form, 364′ 6″ long by 124′ 6″ wide and 246′ 0″ high, and has a storage capacity of 3,100,000 bushels (see Fig. 215). The rectangular "house" is surmounted by a longitudinal superstructure or cupola. The house contains a basement floor and a main floor, and above the main floor is filled with storage bins. The cupola contains the bin floor, a spout floor, a scale floor, and a machinery floor. On the receiving side of the elevator there rise 9 towers which are continuations of the receiving legs. On the opposite sides are 9 shipping legs, and between the two rows of receiving and shipping legs there are 9 cleaning legs. The towers and the exterior walls and the floors of the house and cupola are of steel frame construction, and are covered with steel roof trusses.

Foundations.—The elevator structure rests on a pile and concrete foundation. The piles were sunk by a water jet and a 4,200-lb. hammer, through a mixture of sand and clay, to a depth of from 25 to 45 feet, where hardpan was struck. The lower timbers of the grillage are 8″ × 12″ white pine timbers spaced 2′ 6″ center to center, while the upper layer are of 6″ × 6″ timbers spaced close together. On top of the grillage were built the concrete pedestals to carry the columns. The concrete was made of 1 part Portland cement, 2½ parts clean, sharp sand, and 5 parts Kettle River sandstone. The maximum allowable pressure on the concrete was 500 lbs. per sq. in., and the maximum load on a pile was 22 tons.

Steel Framework.—The rectangular bins are carried on girders, which are in turn carried by 280 Z-bar columns about 42 feet long, arranged as shown in Fig. 217. The Z-bar columns rest on cast bases which are anchored to the concrete pedestals. Details of the columns are shown in Fig. 218. The bin corners are directly above the columns and carry the columnar structure to the level of the bin floor where the columns supporting the upper part of the house begin, as shown

*Engineering News, Sept. 26, 1901.

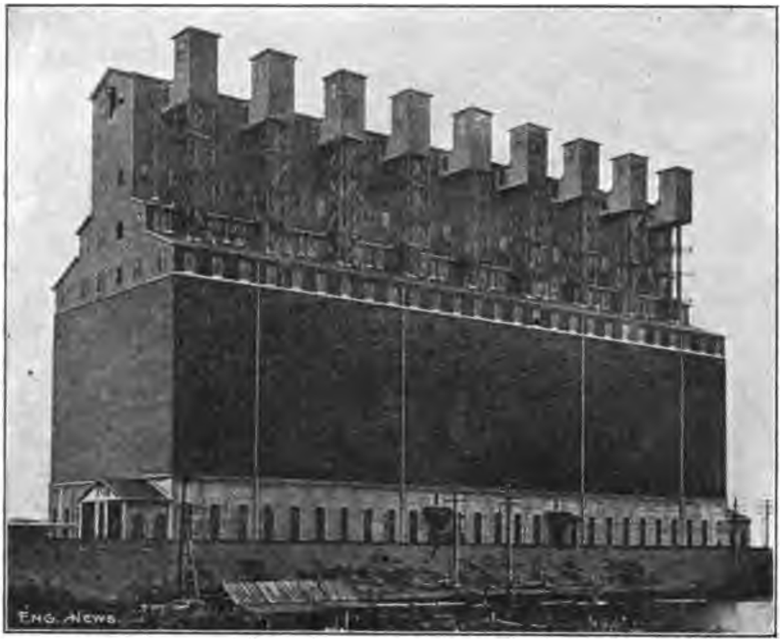


FIG. 215. GENERAL VIEW OF 3,100,000 BUSHEL GRAIN ELEVATOR FOR GREAT NORTHERN RY., AT WEST SUPERIOR, WIS.

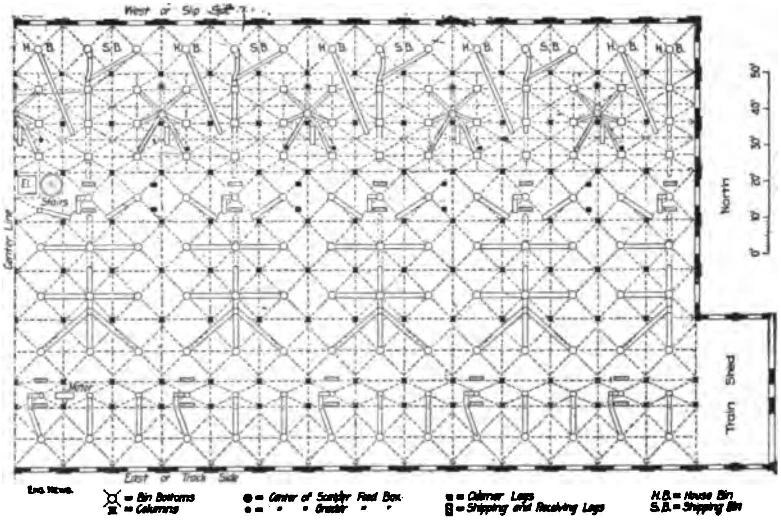


FIG. 216. HALF PLAN AT MAIN FLOOR.

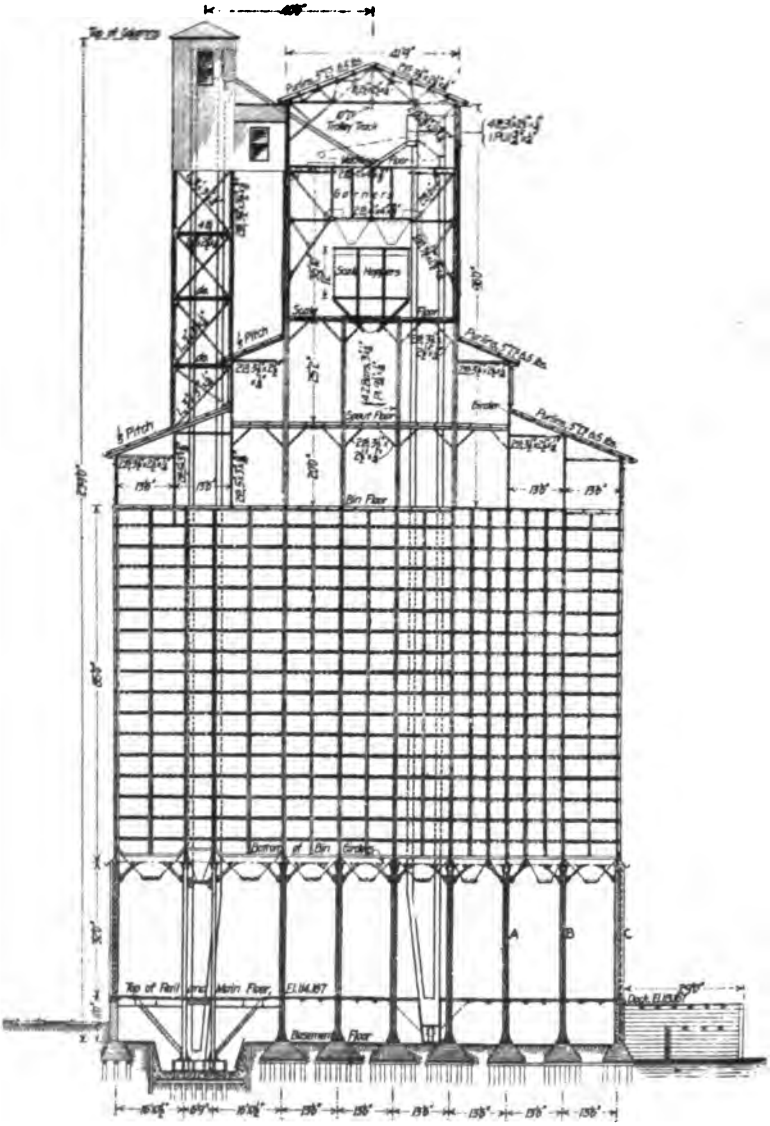


Fig. 217. Transverse Section through Elevator.

in Fig. 217. To the height of the bottoms of the bins the exterior walls are of brick, and above this level they are of corrugated iron.

Bins.—The building contains 505 rectangular bins 85 feet high, divided as follows:

	Bushels.
8 shipping bins, 13′ 6″ x 27′ 0″	24,900
91 standard bins, 13′ 6″ x 13′ 6″	12,800
54 large bins, 16′ 10½″ x 13′ 6″	15,600
28 small bins, 6′ 9″ x 13′ 6″	6,300
324 house and cleaner pocket bins, 6′ 9″ x 4′ 6″	2,100

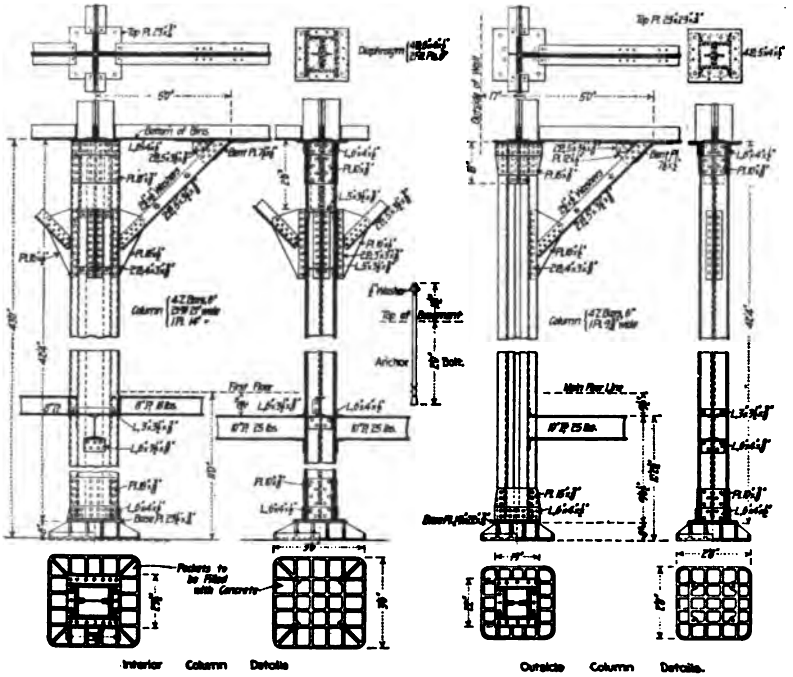


FIG. 218. DETAILS OF COLUMNS SUPPORTING BINS.

The bins are constructed of plates and angles, there being 17 courses of plates 5′ 0″ wide. Starting from the bottom the thickness of plates are 2 courses $\frac{5}{16}''$, 4 courses $\frac{1}{4}''$, and the remaining 11 courses are $\frac{3}{16}''$ thick. The horizontal seams of the bin plates are spliced together with splice plates or butt straps. On every second seam, or at intervals of

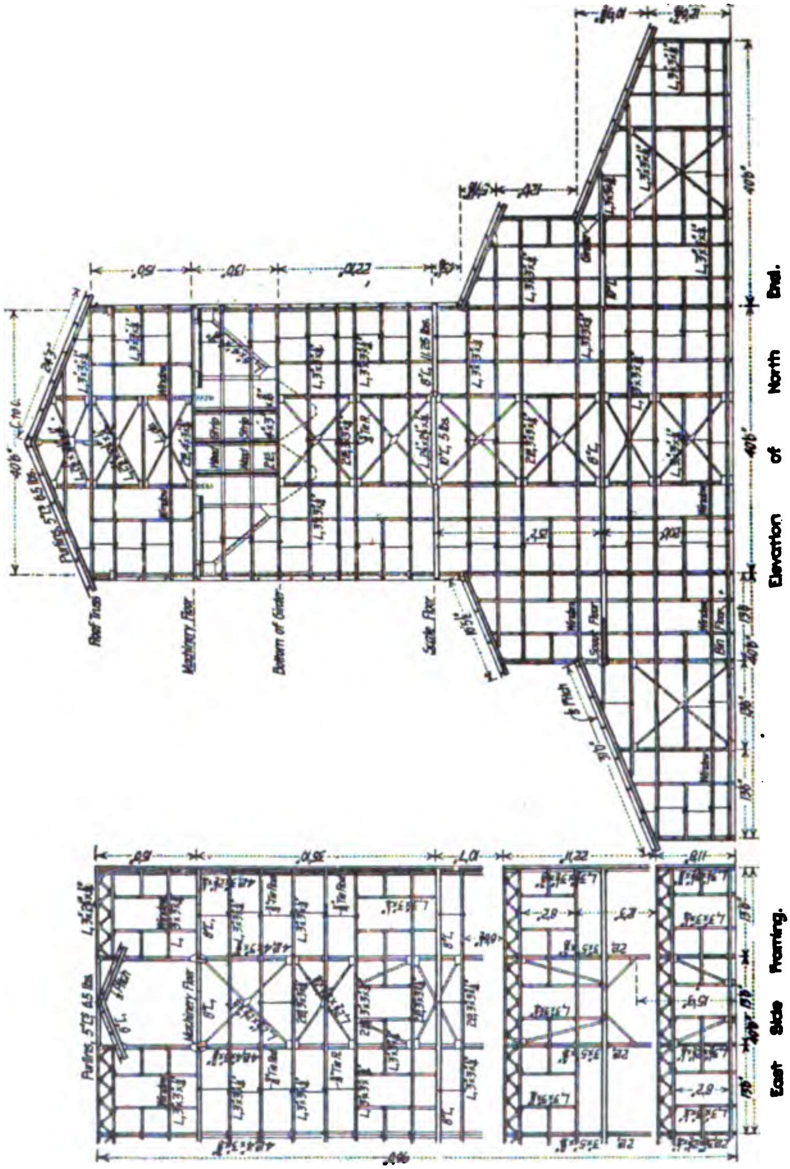


FIG. 219. SECTIONS SHOWING FRAMING ABOVE BIN FLOORS.

10 feet the lower end of these butt straps is bent out toward the center lines of the bin. This construction was adopted for the purpose of arching the grain so that the pressure would be transferred directly to the corners of the bin, thus relieving the pressure on the bin bottom.

The details of the bottom portion of one of the 13′ 6″ × 13′ 6″ bins are shown in Fig. 221. The column and bin construction are shown in Fig. 220. The arrangement of the bins is shown in Fig. 216. Above the bins are four floors and below them two floors, as shown in Fig. 217.

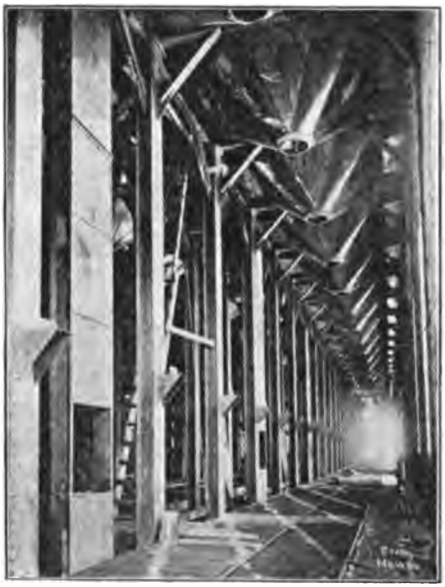


FIG. 220. INTERIOR VIEW ON MAIN FLOOR SHOWING COLUMN AND BIN CONSTRUCTION.

Basement Floor.—On the basement floor are the nine receiving hopper pits, into which grain is discharged from the cars. These pits are concrete lined and into them extend the hopper-shaped boot legs of steel plates. The receiving elevator legs extend up through the bins and are then continued in steel frame towers to the level of the top floor of the cupola. The cleaning leg hoppers do not project below the floor line and the cleaner legs also reach from the basement to the top floor of the cupola, but do not require separate towers.

Main Floor.—The main floor is 11 feet above the basement floor, and is constructed of concrete over brick arches carried by I-beams. This floor contains the dust collecting machinery, car shovels, unloading trucks, etc. A representative section is shown in Fig. 217. Loaded cars are run into the house and to the transfer table located 600 feet from the house by a car haul. There are 18 receiving hoppers 12′ 0″ × 35′ 0″, two being provided for each receiving leg. To unload there are 18 movable power shovels. Details of these power shovels are shown in Fig. 224.

The dust collection system is unusually elaborate and gathers dust from all floors and delivers it into the furnaces in the boiler rooms.

Cupola Floors.—The cupola floors contain four floors, as shown in Fig. 219: (1) The bin floor. (2) The spout and conveyor floor with two 40-in. conveyor belts. (3) The scale floor, containing 18 scale hoppers having a capacity of 1,000 bushels each, and 18 Fairbank's scales with a capacity of 120,000 lbs. each. Above the scale hoppers are 18 scale garners and 9 cleaner garners. The capacity of the garners is 25 per cent larger than the capacity of the scale hoppers. (4) The machinery floor, which contains all the motors which drive

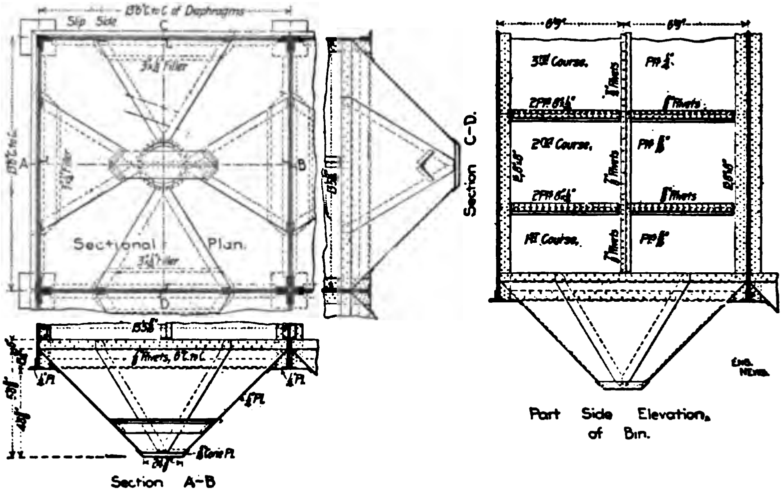


Fig. 221. Details of Bottom of Steel Bin.

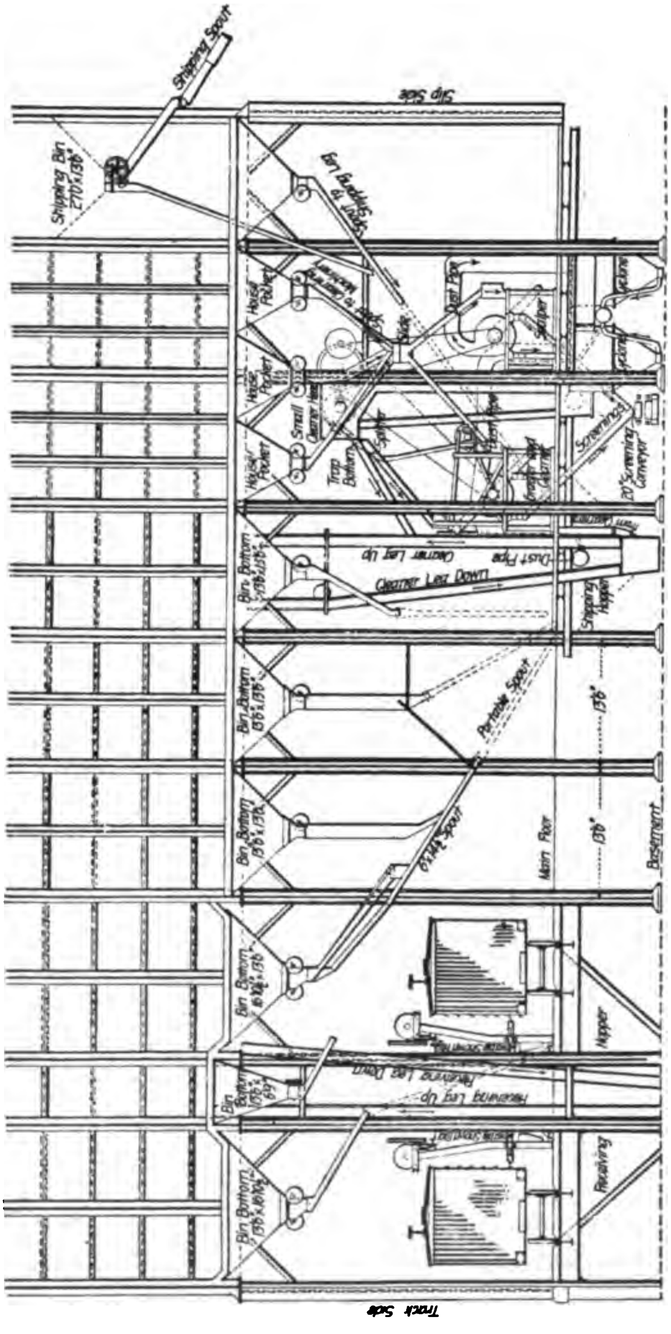


FIG. 222. TRANSVERSE SECTION THROUGH MAIN FLOOR, SHOWING SPOUTING AND MACHINERY.

the 9 receiving, 9 shipping and 9 cleaner legs. The capacity of the receiving and shipping legs is 12,000 bushels per leg per hour, and the capacity of the cleaner legs is 3,000 bushels per leg per hour. The screenings from the machines are discharged on a 20-in. conveyor belt in the basement, which empties into the boot of the cleaner leg.

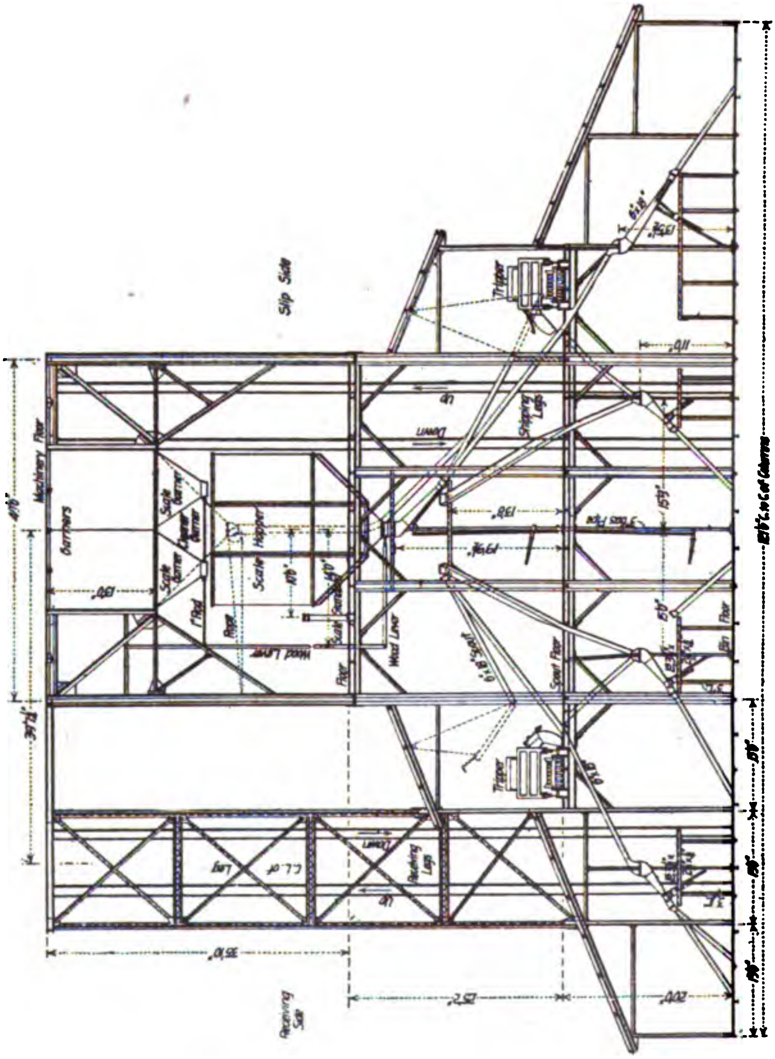


FIG. 223. Transverse Section Through Cupola.

Operation.—When wheat is brought into the elevator in cars, of which nine can be pulled in by a grip cable at one time, it is pushed by the automatic shovels out of the car onto a grating over one of the nine receiving hoppers. Running to the bottom of these are endless belts carrying small buckets which scoop up the grain and hoist it to the very top of the house, the upper part of each belt running in

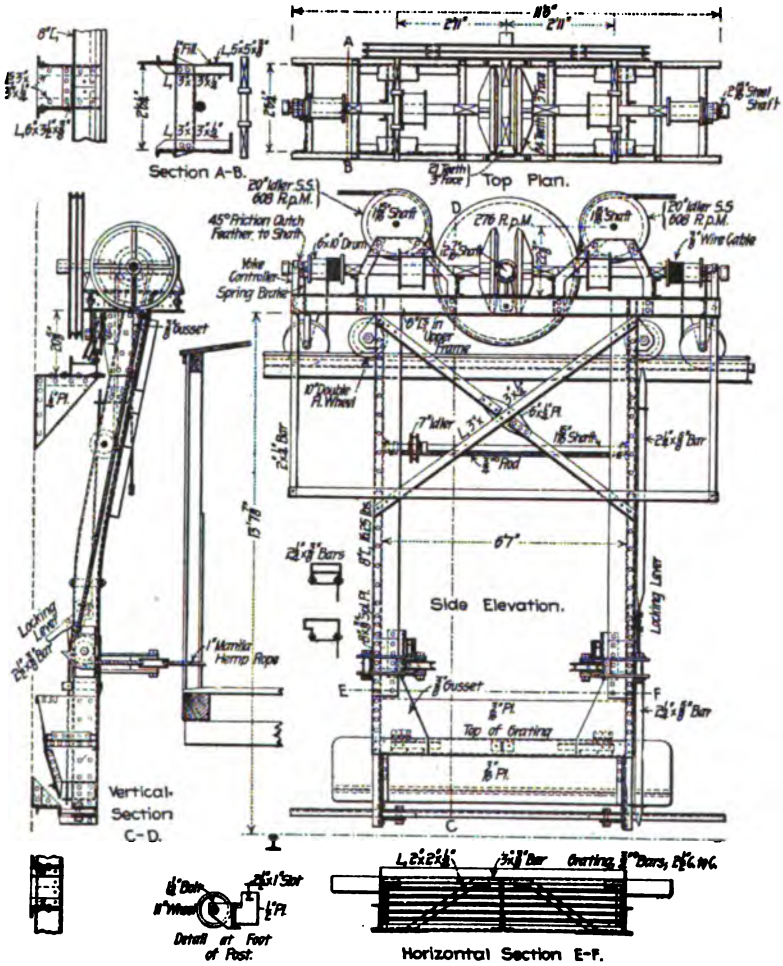


FIG. 224. Details of Power Shovels for Unloading Cars.

one of the nine steel towers. Dumped from the belt the grain is shot into a spout and directed across the top of the building to the garners, from which it falls into the scales. It then drops through a swinging turn-head into any one of numerous spouts and is conducted perhaps to a belt conveyor by which it can be handled longitudinally of the building, perhaps by way of another series of turn-heads and spouts to its bin. If it is to be cleaned it falls through the cleaner bin to one of the cleaning machines and passes over the sieves on the main floor, whence it returns again to the top of the house and down into a bin. If it is to be shipped it is sent a second time to the scales and thence to the shipping bins, which have a capacity sufficient to load a ship of 160,000 bushels at one draft. All screenings from the cleaner are directed to a belt in the basement and then collected. The dust is gathered and conveyed to furnaces in the boiler house.

Materials and Quantities.—The following is a summary of materials used in the construction of the elevator:

Foundation and Walls in Main Story:

Number of piles driven	4,570
Timber and sheet piling, ft. B. M.	380,000
Excavation, cu. yds	23,000
Masonry, cu. yds	1,500
Concrete, cu. yds	3,000
Cut stone, cu. ft	1,300
Brick work, cu. ft	45,000

Superstructure:

Structure below bins, consisting of columns, knee braces, car and shipping hoppers, grillage beams and girders, etc., tons	1,850
Bins proper, consisting of bin plates, corner angles, tie bars, bin bottoms, etc., tons	6,500
Cupola material, tons	1,450
Legs and spouts below bin floor, tons	450
Legs and spouts above bins manufactured at elevator site, tons	350
Total tons	10,600
Total length of legs and spouts (nearly) 8 miles.	
Total number of field rivets	2,000,000

Machinery:

1 20″ screening conveyor, in basement.
2 40″ conveyors, on spout floor.
24 cleaners, on main floor.
12 flax machines, on main floor.
18 movable power shovels, on main floor.
9 receiving legs.
9 shipping legs.
9 cleaner legs.
16 marine loading spouts.
18 120,000-lb. scales.
42 3-phase 60 cycle, 440 volt electric motors, total 2,122 H. P.

Lighting:

1,000 16-candle-power incandescent lamps.

Cost.—The following unit costs were furnished the author by Max Tolz, mechanical engineer, Great Northern Ry., who had charge of the design and erection of the elevator:

Piles cost 10 cents per lineal foot.

Driving 40-ft. piles, $1.00 each.

Concrete cost $6.25 per cu. yd.

Structural steel work cost $50.00 per ton f. o. b. Pittsburg, Pa.

Erection of structural steel work, including the driving of all rivets, $13.25 per ton.

The total cost of the elevator was 39.65 cents per bushel of storage.

The costs, especially the cost of structural steel and erection, are very low.

The work was begun May 1, 1899; the erection of steel January 1, 1900; the first car of grain was received on February 26, 1901.

GRAIN ELEVATOR, SANTA FE SYSTEM, CHICAGO.

General Description.—The elevator comprises a frame working house of 400,000 bushels capacity, with car shed, marine tower, 35 reinforced concrete storage bins, having a total capacity of 1,000,000 bushels, a drying and bleaching equipment, and a 1,500 H. P. power plant. A general view of the elevator is shown in Fig. 225, and a plan is given in Fig. 226. This elevator was designed by the John S. Metcalf Co., Chicago.

Working House.—The working house is 225 feet long and 56 feet wide, and is of timber cribbed construction, covered on the outside up to the top of the bins with brick, and above that point with corrugated steel. There are 5 receiving and 5 shipping legs, having a capacity of 15,000 bushels per hour each, making it possible to receive and ship 75,000 bushels per hour. In addition to these there are 15 smaller legs for serving the five clippers, five cleaners, one screener, and the drying and bleaching plants; also one leg for the disposal of screenings from the machines.

In the cupola are 10 scale hoppers, each of 1,600 bushels capacity. In addition to the above are a dust collecting system, a large car pulley, a longitudinal conveyor in the cupola for conveying grain lengthwise of the house, a passenger elevator, etc. There is a marine tower with a capacity of 6,000 bushels per hour; also eight vessel loading spouts. The arrangement of the bins and the elevating machinery is shown in Fig. 227. The working house bins are made of timber, with steel hopper bottoms and gates. Elevator legs and heads throughout are of steel, and most of the spouts are of steel.

Loading and Unloading.—A steel shed extends over four tracks, as shown in Fig. 227. On each track are five unloading places spaced 45-foot centers, in order to accommodate the longest cars. Beneath each unloading place is a steel track hopper with a capacity of a full car load. Each hopper discharges through four openings to a 15,000 bushel conveyor running below a line of four hoppers, across the basement under the track shed to one of the receiving legs. It is so arranged that but one of the four hoppers can be discharged into the conveyor at one time. Ten car loading spouts are arranged to load cars at five places on two tracks. The design of the working house is unusual in having the cars unload in an unloading shed.

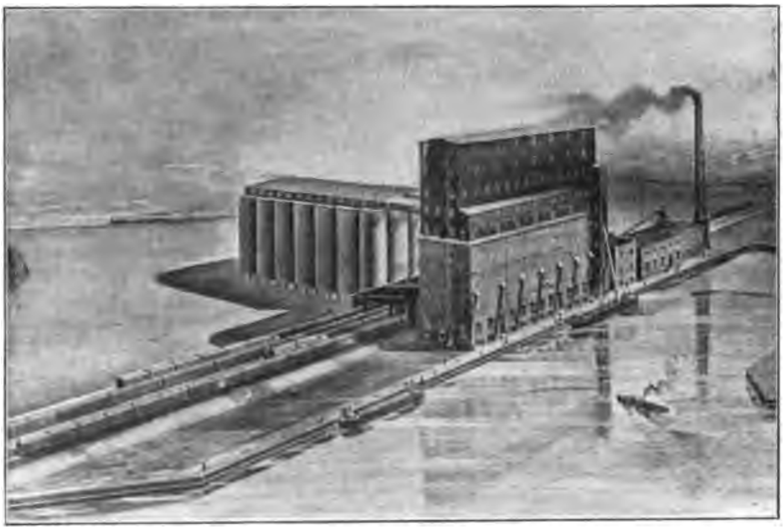


FIG. 225. GRAIN ELEVATOR FOR SANTA FE RY., CHICAGO, ILL.

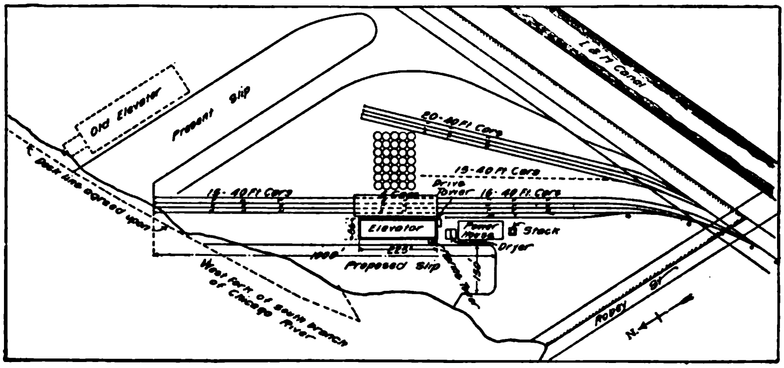


FIG. 226. PLAN OF ELEVATOR AND TRACKS.

Concrete Bins.—The reinforced concrete storage bins are 35 in number, and are 23 feet inside diameter and 80 feet high. In addition to this there are 24 interspace bins. The detail plans for the bins are given in Fig. 228. The conveyors have a capacity of 15,000 bushels per hour for filling and emptying the tanks. The detailed specifications follow:

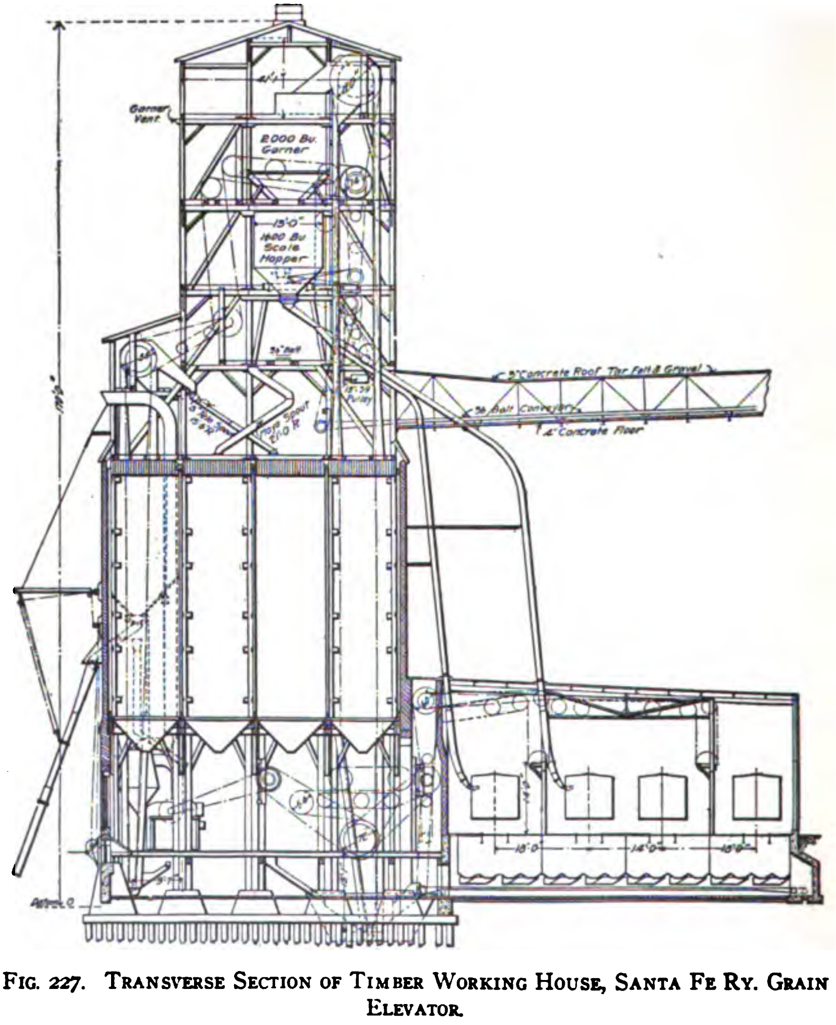


FIG. 227. TRANSVERSE SECTION OF TIMBER WORKING HOUSE, SANTA FE RY. GRAIN ELEVATOR.

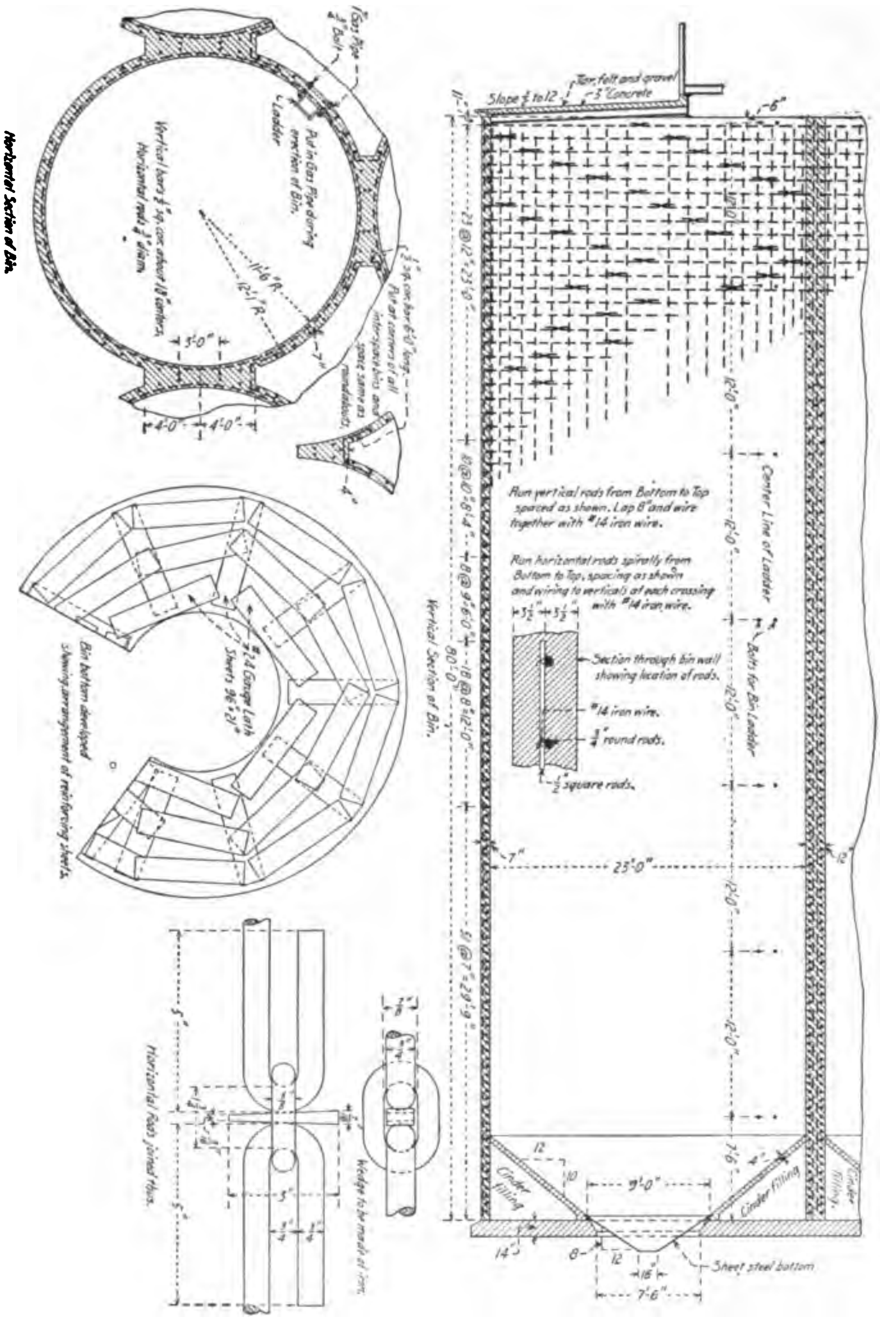

Horizontal Section of Bin.
1" Gas Pipe
3/4" Bolt
Put in Gas Pipe during erection of Bin.
Ladder
Vertical bars 3/4" sq. cor. about 18" Centers.
Horizontal rods 3/4" diam.
11'-6" R.
12'-1" R.
7"
3'-0"
4'-0"
4'-0"
1" sq. cor. bar 6'0" long. Put at corners of all interspace bins and space same as roundabouts.
4"
#24 Gauge Lath Sheets 96" x 21"
Bin bottom developed showing arrangement of reinforcing sheets.
Horizontal Rods joined thus.
Wedge to be made of iron.
5"
5"
3"
3/4"
3/4"
Vertical Section of Bin.
Tar, felt and gravel
3" Concrete
Slope 3/4 to 12
11"
6"
23 @ 12" = 23'0"
10 @ 10" = 8'4"
8 @ 9" = 6'0"
18 @ 8" = 12'-0"
51 @ 7" = 29'9"
80'0"
12'0"
Center Line of Ladder
Bolts for Bin Ladder
Run vertical rods from Bottom to Top spaced as shown. Lap 8" and wire together with #14 iron wire.
Run horizontal rods spirally from Bottom to Top, spacing as shown and wiring to verticals at each crossing with #14 iron wire.
3 1/2"
3 1/2"
Section through bin wall showing location of rods.
#14 iron wire.
3/4" round rods.
1/2" square rods.
7"
23'0"
12"
7'6"
4"
12
10
9'0"
Cinder filling
Cinder filling
Cinder filling.
14"
8"
12
16"
7'6"
Sheet steel bottom

FIG. 228. DETAILS OF REINFORCED CONCRETE GRAIN BINS, SANTA FE RY. GRAIN ELEVATOR.
]

DETAILED SPECIFICATIONS FOR CONCRETE STORAGE BINS.*

CONCRETE BINS.—Thirty-five cylindrical concrete bins, 23′ 0″ in diameter inside, shall be built as shown on the drawings. The interspaces between these bins shall also be used for the storage of grain. The walls shall be of concrete of the ingredients hereinafter specified.

Concrete.—Concrete shall be composed of the broken stone or gravel, sand and cement hereinafter specified. A batch mixing machine approved by the chief engineer may be used. The ingredients shall be placed in the machine in the volumes specified, and sufficient water added to form a mixture fluid enough to give clean, smooth surfaces when deposited in the forms. The mixing shall be continued until the mass is uniform; it shall be done as rapidly as possible and the batch deposited in the work without delay. If the mixing is done by hand, it shall be performed in a manner satisfactory to the chief engineer.

The concrete in the bin walls and in the hopper bottoms shall be mixed in the following proportions: One measure of Portland cement, not more than 2½ measures of sand, and not more than four measures of broken stone; and the proportions of sand and stone shall be reduced if necessary, so that in all cases the sand will thoroughly fill the voids in the stone and the cement will thoroughly fill the voids in the sand. Cement shall be measured in the original packages. The sand and broken stone must be actually measured in bulk. No counting of shovelsful or other approximations will be allowed.

All concrete shall be mixed in small and convenient quantities and immediately deposited in the work. It must be carefully placed, and not dropped from sufficient height to separate the mortar from the broken stone. It shall be laid in sections and in horizontal layers. It shall be deposited in a state sufficiently fluid to form walls which are smooth and free from voids.

In no case shall concrete be permitted to remain in the work if it has begun to set before the ramming is completed. Layers of concrete must not be tapered off in wedged-shaped solids, but must be built with square ends. The concrete must be so thoroughly compact that there shall be no pores or open spaces between the stone of which it consists that are not thoroughly filled with mortar.

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* These specifications were prepared by the John S. Metcalf Co., Chicago, Ill., who has kindly granted permission to have them published.

The walls during the progress of the work shall be kept as near a uniform height as possible. Before depositing any concrete after work has been discontinued for over one hour, the forms shall be cleaned of any concrete adhering to them, and all fins or broken pieces on the top of the wall removed; and the forms and upper surface of the concrete shall be thoroughly wetted. On all exposed surfaces of concrete, a wedge-shaped rammer shall be used to force the stone back and leave a clean cement-mortar face, without voids, on the surface. Before any weight is placed on the concrete, it shall have as much time to set as can conveniently be allowed, and in no case less than 24 hours. Concrete laid in cold weather shall be protected from freezing.

Pointing.—After the molds are removed, if there should be found any small pits or voids on the exposed surfaces of the concrete, such porous places shall be neatly stopped with pointing mortar. The mortar shall be composed of one part cement and one part sand. This mortar shall be mixed in small quantities and used before it shall commence to set. No plastering of any surfaces will be allowed.

Finishing Outside Walls.—After the walls have been carried to the full height, the contractor shall go over the entire outside surface with a cement wash evenly put on with a brush, leaving the entire surface of the bin walls of a uniform color.

Broken Stone.—The broken stone used in this work shall be so crushed that it will pass through a 1″ ring. All stone used shall be a good hard quality of limestone, free from clay, dust, or other impurities, but the smaller particles of stone that come from the crusher shall be left in. Clean washed gravel, so crushed as to pass through a 1″ ring, may be substituted for the above stone on the written consent of the chief engineer.

Sand.—All sand used in this work shall be clean, coarse, sharp river, bank, or lake shore sand, free from all loam or vegetable matter, and shall be subject to the approval of the chief engineer.

Portland Cement.—All Portland cement will be furnished by the company, f. o. b. cars at the elevator site, for all of the building. It shall be of such quality as to conform to the standard specifications for Portland cement as adopted November 14, 1904, by the American Society for Testing Materials. Tensile tests shall not fall below the lower limit of minimum tensile tests as given in the specifications referred to. If the cement furnished fails in any particular to conform to the above-mentioned specifications it may be rejected. All cement shall be kept housed and dry by the contractor until wanted in the work. The contractor shall furnish the chief engineer a report of the tests of the cement in each car supplied, which report shall be made by a competent engineer and shall not be a factory test. Reports shall be furnished to the chief engineer sufficiently in advance of using the cement so that he may give his approval of at least the seven-day tensile tests before the material is used. Factory tests will be accepted if the contractor obtains express written permission from the chief engineer to that effect.

Molds for Concrete.—The contractor shall furnish and construct all necessary molds to form the concrete to the exact sizes shown on the drawings. They shall be of a substantial character, made of planks dressed one side and two edges, or of dressed and matched flooring, and the frames holding them shall be of sufficient strength so they will be practically unyielding during the process of filling and tamping. The forms shall be so constructed and manipulated that the walls of the bins will be perpendicular and free from ledges or pockets upon which grain could lodge, and when the molds are used more than once they shall be thoroughly cleaned before using a second time. All surfaces of forms which come in contact with the concrete shall be treated with soap or crude oil before the concrete is deposited. When the forms are ready to be filled with the concrete, the chief engineer shall be notified, and no concrete shall be deposited in the forms until he has inspected the arrangement of the steel and the various fastenings of the reinforcing rods, and given his approval of the work.

Reinforcing Rods.—The horizontal reinforcing rods shall be round rods made of mild merchantable steel, ¾″ in diameter. These rods shall be bent to the curve of the bins and shall have their ends hooked, and be connected by links, held securely in place by wedges, in the manner indicated on the drawings. The several bands of horizontal rods shall be arranged in a spiral, running continuously from the bottom to the top of the wall. The couplings of the rods shall break joints with each other, and the bands shall be spaced as shown on the drawings.

The vertical rods shall be ½″ corrugated square steel bars spaced as shown on the drawings. The vertical rods shall be not less than 12′ 0″ long except at the starting and the finishing of the wall. The ends shall be wired together, lapping them 8″. Adjacent rods shall be arranged to break joints with each other. The crossings of the horizontal and vertical rods, and the ends of the vertical rods, shall be properly and securely fastened together with No. 14 annealed wire, as shown on the drawings, or with steel clips, samples of which have been submitted to the chief engineer, and have received his approval.

All other reinforcing rods shall be put in place wherever necessary or wherever shown on the drawings.

Corrugated Bars.—The vertical bars in the bin walls shall be corrugated square steel bars known as "Johnson" bars. Other square reinforcing bars shall be "Johnson" bars or "Ransom" bars.

Anchor Bolts.—Anchor bolts shall be set in the concrete as shown on the drawings for attachment of structural steel, etc.

Bin Ladders.—Each bin in the storage annex shall be supplied with a steel ladder, erected at the location shown on the drawings, and painted with a good coat of oxide of iron paint before erection.

THE CANADIAN PACIFIC GRAIN ELEVATOR, PORT ARTHUR, ONTARIO.

General Description.—The elevator consists of a steel working house 16′ 0′ × 36′ 0′ and 166′ 0″ high, which contains the working machinery, and a reinforced concrete grain storage consisting of 9 circular reinforced concrete grain bins 30 feet in diameter and 90 feet high, with a capacity of 43,500 bushels each, and four interspace bins, with a capacity of 13,100 bushels each, making a storage capacity of 443,000 bushels. The working house is connected with the drying plant by a three-span steel conveyor bridge 230 feet long, carried on steel towers 60 feet high. The conveyor gallery contains a 36-inch reversible belt conveyor. An elevation of the bins is given in Fig. 229.

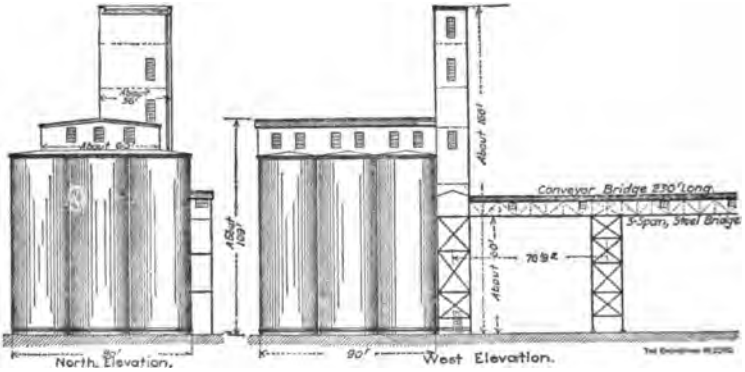

FIG. 229. ELEVATION OF CONCRETE GRAIN BINS.

Machinery.—The equipment includes an elevator leg with a capacity of 15,000 bushels per hour; two 36-inch belt conveyors over the bins; three 36-inch belt conveyors under the bins to receive grain from them and one 36-inch belt conveyor transverse to the three to carry grain to the leg; three garner weighing bins and shipping scales. The elevator machinery is run by a rope drive from an engine house.

Bins.—The reinforced concrete bins, Fig. 230, are 30 feet in diameter, 90 feet high and have walls 9 inches thick carried on foundations 24 inches thick. The concrete was made of 1 part American Portland cement, 3 parts sand, and 5 parts Lake Superior gravel. The concrete was reinforced with steel bars placed horizontally and vertically. The horizontal reinforcement consisted of pairs of steel bars, one bar near each surface of the wall, and placed 12 inches apart vertically. For the first 15 feet above the base the steel had an area of 1 sq. in. per foot of height (2–2″ × $\frac{1}{4}$″ bars); for the next 35 feet an area of 0.88 sq. in. per foot (2–1$\frac{3}{4}$″ × $\frac{1}{4}$″); for the next 20 feet an area of 0.75 sq. in. per foot (2–1$\frac{1}{2}$″ × $\frac{1}{4}$″); for the next 20 feet an area of 0.50 sq. in. per foot (2–1″ × $\frac{1}{4}$″); and above this 0.38 sq. in. per foot (2–1″ × $\frac{3}{16}$″). Besides the horizontal bars there are 27 vertical bars $\frac{1}{2}$-inch diameter equally spaced. Where the walls are thickened at the contact points of tangencies they are reinforced by horizontal layers of 2-inch

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*Engineering Record, April 9, 1904.

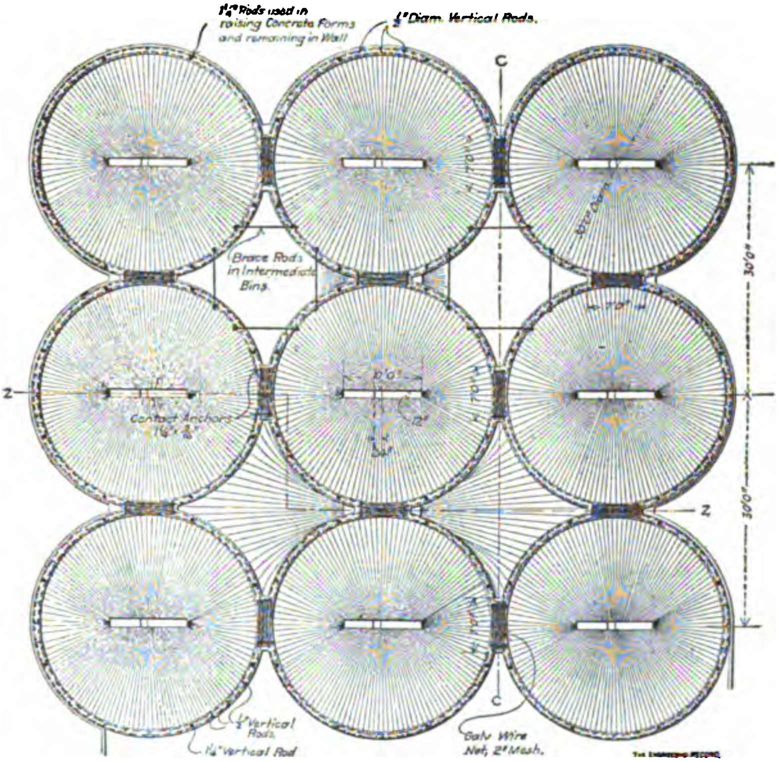


FIG. 230. HORIZONTAL SECTION THROUGH BINS, SHOWING INTERMEDIATE BINS AND HOPPER BOTTOMS.

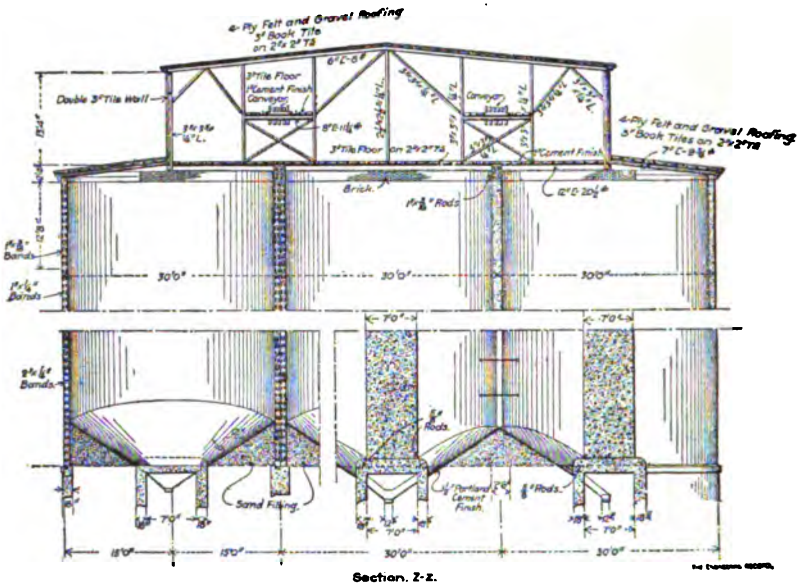


FIG. 231. TRANSVERSE SECTION THROUGH CIRCULAR AND INTERMEDIATE BINS.

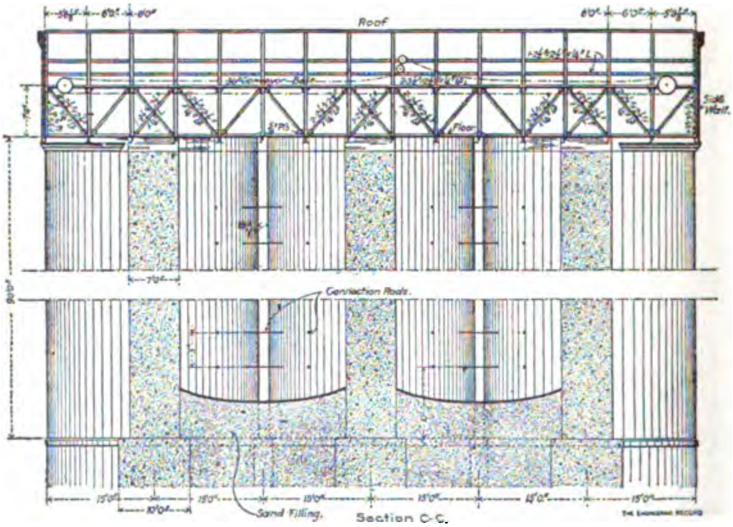


FIG. 232. LONGITUDINAL SECTION THROUGH INTERMEDIATE BINS.

mesh galvanized netting placed 12 inches apart vertically. Also by 2 horizontal 1¼ × $\frac{5}{16}$-inch anchors 12 inches apart vertically, which hook over the bars.

The interspace bins are braced by tie bars, as shown in Fig. 230, spaced 5 feet apart vertically. These bars have a diameter of 1¾ inches up to 20 feet high; 1⅝ inches for the next 30 feet; 1½ inches for the next 20 feet; and 1⅛ inches to the top. The ends of adjacent bars are connected by a horizontal steel band.

The floors of the bins are made of 3 inches of concrete placed on a

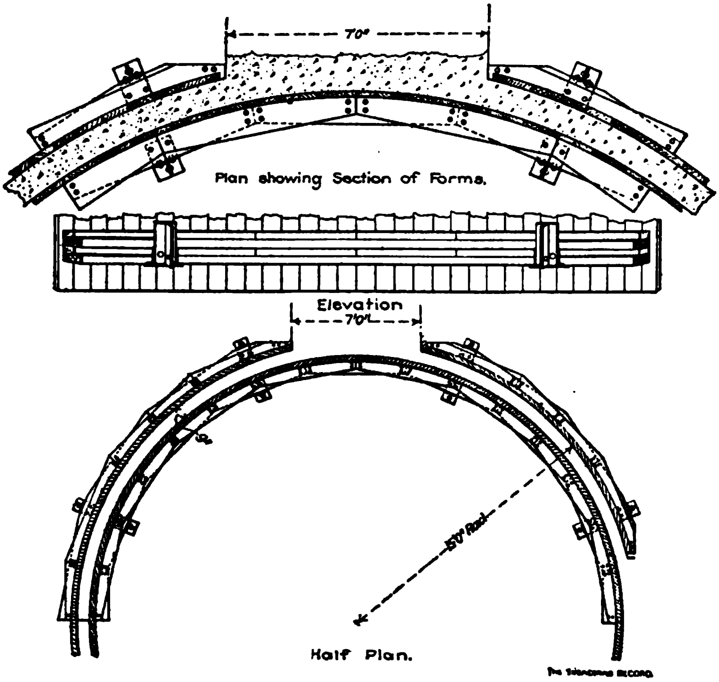
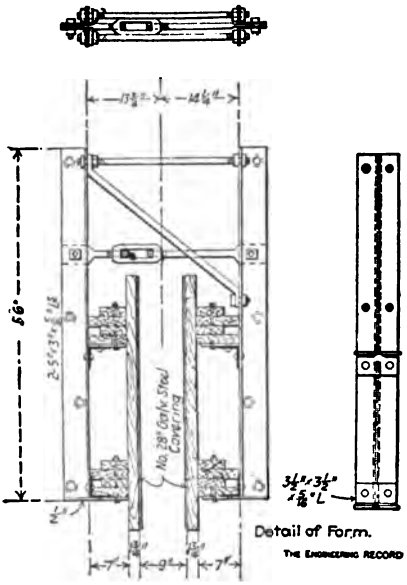


Fig. 233. Plan of Forms for Circular Bins.

sand filling, and 1½-inch wearing Portland cement coat. Additional details are given in Figs. 231 and 232.

Construction of Concrete.—The walls of the bins were made in movable cylindrical forms 4 feet high, as shown in Fig. 233 and Fig.

234. The curved surfaces of the forms were made of 2-inch vertical planks spiked to inside and outside circular horizontal chords. The chords were made like ordinary arch centers with four thicknesses of 2·× 8-inch scarf planks bolted together to break joints and make complete circles inside the bins and circular segments of 270 degrees or less on the outside surface of the bins. The molds were faced on the inside with No. 28 galvanized steel and were maintained in concentric positions with fixed distances between them by means of eight V-shaped steel yokes in radial planes. Each yoke consisted of an inside and an outside vertical post with a radial web and flanges engaging the inner and outer faces of the circular chords. The posts projected about 2 feet above the tops of the molds and were rigidly connected there by means of heavy braces and an adjustable tension bar. The yokes were bolted to the inner and outer chords of the molds and vertically united them into a single structure. The lower ends of the vertical yoke posts were seated on jack-screws which were supported on false



Detail of Form.

THE ENGINEERING RECORD

FIG. 234. DETAIL OF FORMS FOR CIRCULAR BINS.

work built up inside the tanks as the work progressed. The inside surface of the mold was a complete cylinder, but the outer surface was made in 2, 3 or 4 sections, to allow for the connections between the concrete walls at the points of tangency of the different tanks.

Steel Construction.—The steel framework on the tops of the bins is shown in Figs. 231 and 232, except the conveyor bridge which consisted of three 61′ 8″ spans composed of two connected trusses 8′ 4″ deep and spaced 10′ 0″ center to center. The construction is similar to the trusses over the bins, as shown in Fig. 232.

The steel garner bin is 13′ 0″ square and 5′ 0″ deep, with W-shaped bottom, and is made of $\frac{1}{4}$″ plates and 3″ × 3″ × $\frac{1}{4}$″ angles. The steel scale hopper is 12′ 0″ square and 9′ 0″ deep, with a pyramidal top, and is made of $\frac{3}{16}$″ plates. The steel shipping bin, 13′ 0″ × 15′ 0″ is made of $\frac{1}{4}$″ plates.

The total weight of the steel work in the elevator is 400,000 pounds.

The elevator was designed by the engineering department of the Canadian Pacific Ry., Mr. E. H. McHenry, chief engineer, and was constructed by the Barnett-Record Company. The contract was let January, 1903, and the elevator was ready for service January 3, 1904.

The Canadian Northern Railway Tile Grain Bins, Port Arthur, Ontario.

General Description.—The grain storage consists of 80 circular tile grain bins arranged as shown in Fig. 235, Fig. 236, and Fig. 237,

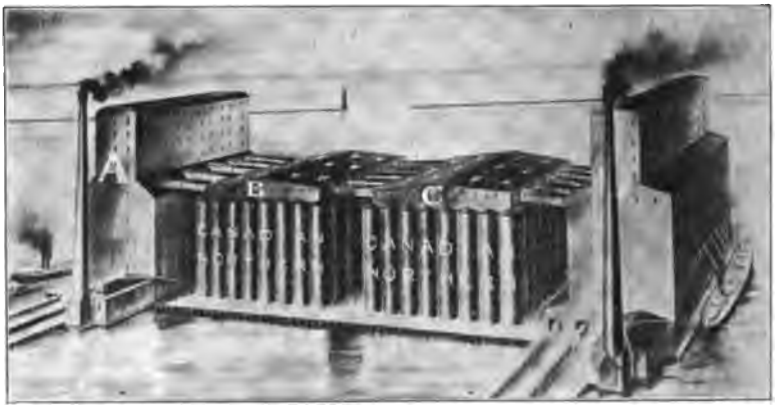


Fig. 235. Grain Elevator for Canadian Northern Railway, Port Arthur, Ontario. (The Original Elevator "B" is Described Here.)

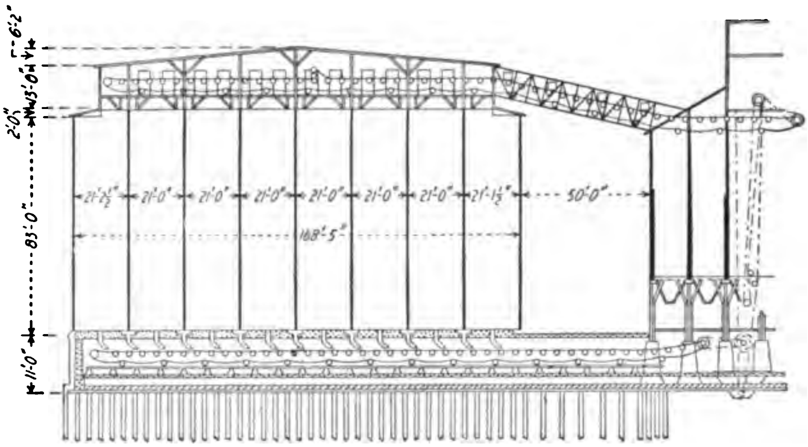
and located 50 feet south of the working house. The bins are each 21′ 0″ outside diameter and 83′ 0″ from the floor to the eaves, and are

The capacity of the bins is as follows:

	Bushels
48 circular bins, each 21,825 bu	1,047,600
32 circular bins, each 21,440 bu	686,080
63 intermediate bins each 6,300 bu	396,900
Total capacity of bins	2,130,580

The tile bins rest on a pile and concrete foundation surrounded by a retaining wall on three sides of the building to protect it from the waves. The piles are of pine, spruce, and tamarac, 40 to 50 feet long, 12 and 8 inches in diameter at the butt and top, respectively. They were driven so that there was not to exceed one inch penetration for a 2,000-lb. hammmer dropping 20 feet. The concrete was composed of 1 part Portland cement, 3 parts clean gravel, ranging in size from the size of a pea to that which would pass a 2½-inch ring. The floors were made of reinforced concrete, as shown in Fig. 238.

Bin Walls.—The walls of the tanks were constructed of Barnett-Record semi-porous tile, constructed as shown in Fig. 201. The main tiles were 12" × 12" × 5", alternating with channel tiles 12" wide, 3" deep and 5" thick, laid as shown. The outside tile were hard burnt semi-glazed tile 12" × 12" × 12". The facing tiles were firmly anchored to each other and to inner tile in every fourth course where the joints came together with a piece of 3" × 4" galvanized wire netting, four meshes to the inch, one anchor to each tile. The openings in the upper end of the main wall tile were covered with wire netting, three meshes to the inch, to permit of a full mortar bed to receive the channel tile. In the channel tile were placed steel bands, not less than two steel bands in each channel, the weight of the bands decreasing from the bottom to the top. All the tile were laid in Portland cement mortar, composed of 1 part Portland cement and 3 parts clean, sharp sand, tempered with lime mortar. No lime was put in the mortar used to

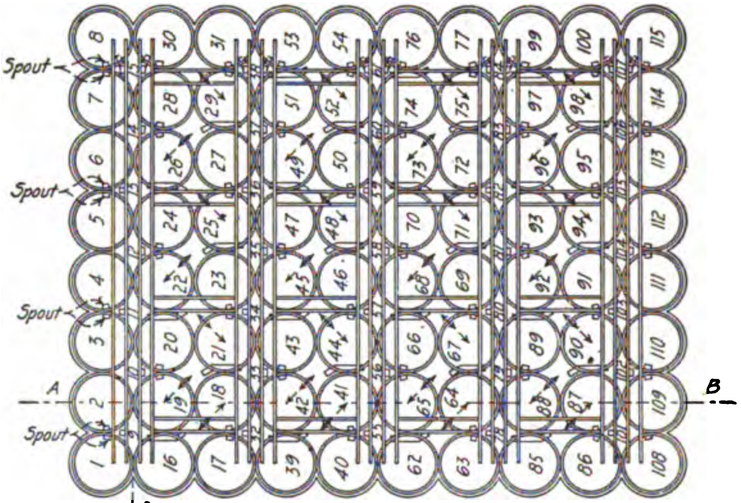


Longitudinal Section through Tanks
2000000 Bushel Fireproof Storage
Canadian Northern Ry.
Port Arthur, Ont.

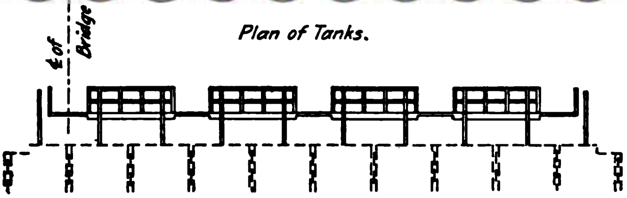
FIG. 236.

imbed the steel bands. Tiles in the cupola walls were of two thicknesses of 3″ hollow tile laid in the same manner as the main wall tiles.

Roof.—All roof surfaces except the bridges and tunnels were covered with 12″ × 18″ × 3″ hollow book tile, laid in a bed of Portland cement mortar on the flanges of tee sub-purlins. On top of the tile thus laid was laid a four-ply composition felt and gravel roofing. The exposed edges of each ply were cemented down with hot pitch and the entire surface was covered with straight-rim medium soft domestic coal tar roofing pitch, 100 lbs. to the square of 100 square feet. The gravel was screened through a ¾-inch wire mesh screen, ⅛ cu. yd. being used to the square.



Plan of Tanks.

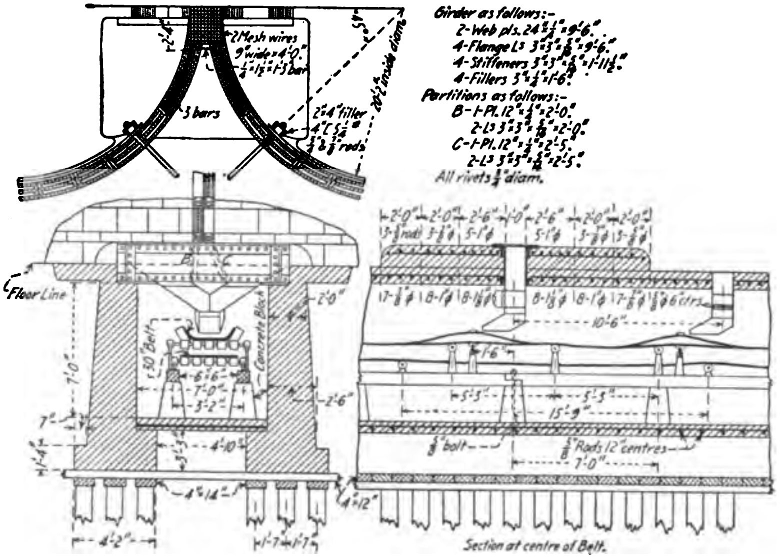


Section A-B.

2000000 BUSHEL FIREPROOF STORAGE
CANADIAN NORTHERN RY.
PORT ARTHUR, ONT.

FIG. 237.

Conveyors.—Each bin coming over the tunnels or conveyors is provided with a cut-off spout with an extension made of No. 14 steel, arranged to load the grain on the belts. In the upper part of the building no spouts are needed except for the intermediate bins, which are provided with spouts of No. 14 steel to take the grain from the tripper spouts.



Section through Tunnel, looking toward old elevator.

2000000 BUSHEL FIREPROOF STORAGE
CANADIAN NORTHERN RY.
PORT ARTHUR, ONT.
Details of Tunnel and Conveyor.

FIG. 238.

There are 5 belt conveyors, with 36-inch belts, running from the working house out over the bins, and supported by disc rolls 5′ 3″ apart. The return belt is carried on straight rolls, 15′ 9″ center to center. The shafts for these rolls are $1\frac{3}{16}''$, with standard oscillating bearings fixed to wood supports resting on the steel framework, the wood support being covered with No. 26 plain sheet steel. The conveyor belts in the basement are 36″ wide, carried as in the upper conveyors. In the upper conveyor belt there is one 2-pulley tripper for each belt. These trippers are provided with self-propelling devices, platform levers, etc., and with spouts discharging to either side, so that the circular bins can be filled without any other spout than that provided by the tripper. These spouts are provided with cut-off gates and arranged with spouts to the intermediate bins. All trippers are carried on 16-lb. tee rails attached to the round supports which carry the rolls.

The capacity of each of the five top conveyor belts is 15,000 bushels per hour, and each of the five lower belts is 10,000 bushels per hour. The conveyor belts are four-ply rubber belting made of 30-oz. cotton duck, pliable with hard surface stretched and spliced with Smith's patent No. 1 belt fastener. These belts are driven by light leather belting. The conveyor head pulleys are covered with four-ply rubber belting. The bridges for each of the five belt conveyors consist of two steel trusses, as shown on the plans; the floor is of book tile and concrete resting on tees and angles as in the upper belt conveyor galleries. The walls of the bridge are covered with No. 24 and the roof with No. 22 galvanized corrugated steel. The bridges are provided with telescope joints to provide against settlement.

Miscellaneous Details.—All woodwork for window and door frames is covered with crimped galvanized iron. The 15 galvanized iron frame skylights in the roof are glazed with ¼" unpolished glass. The cornices and flashing are made of No. 26 plain galvanized iron.

The bin ladders are made of iron bars built into the walls, one ladder to each bin, extending from the top to the bottom. Two fire-escape ladders were also provided on the outside walls.

Painting.—The structural steel work received in the shop one coat of boiled linseed oil mixed with 10 per cent of lampblack and oil. All woodwork around windows received two coats of red lead and oil.

Miscellaneous Data.—Each eight feet in height of each bin required the following materials:

378 tiles, 12" × 12" × 5",
378 channel tiles, 12" × 3" × 5",
216 facing tiles, outer bins only,

336 facing tiles, corner bins only,
250 brick,
375 pieces wire netting, $12'' \times 4''$,
12 pieces wire netting, $4' \times 8''$,
126 pieces wire netting, $12'' \times 1''$, for facing tile,
12 clips, $12'' \times 1\frac{1}{2}'' \times \frac{1}{8}''$,
6 crooked steps, $3'\ 7'' \times \frac{5}{8}''\ \phi$ iron,
6 straight steps, $2'\ 6'' \times \frac{5}{8}''\ \phi$ iron,
4 tie rods, $11'\ 0'' \times \frac{3}{4}''\ \phi$ iron.

Each thousand tiles took $\frac{3}{4}$ cubic yard mortar grout.

Each bricklayer laid 364 tiles per day of 10 hours in the month of November.

Each inside bin has four double channels 75 feet long to carry the rods which support the intermediate bins.

Each outside bin has two double channels 75 feet long.

The hoops or bands in the channel tiles consisted of $1\frac{1}{2}'' \times \frac{3}{16}''$ and $1\frac{1}{2}'' \times \frac{1}{8}''$ steel bars, three complete hoops being used in the lower and two in the upper, thus varying the section to suit the pressure.

Cost.—The entire cost of the building, including revetment and everything complete except the earth filling around and under the foundation was $397,095.80, or 19 cents per bushel of storage capacity.

CHAPTER XX.

COST OF GRAIN BINS AND ELEVATORS.

COST OF STEEL GRAIN ELEVATORS.—For a detailed cost of the different parts of steel grain elevators and for methods of estimating, see Chapter XIII. The following actual costs of steel elevators will give a fair idea of the range of costs.

Cost of Great Northern Ry. Grain Elevator, Superior, Wis.—The following unit costs were furnished the author by Max Tolz, mechanical engineer, Great Northern Ry., who had charge of the design and erection of the elevator:

Piles cost 10 cents per lineal foot.

Driving 40 ft. piles cost $1.00.

Concrete cost $6.25 per cu. yd.

Structural steel cost $50.00 per ton f. o. b. Pittsburg, Pa.

Erection of structural steel work, including the driving of all rivets, $13.25 per ton.

The elevator weighed 7 lbs. per bushel of storage capacity.

The total cost of the elevator was 39.65 cents per bushel of storage. This cost was very low.

For quantities of materials see Chapter XIX.

Cost of Independent Elevator, Omaha, Neb.—The total weight of steel in the elevator was 1,700 tons; the steel in the working house weighing 900 tons and the steel in the circular bins and conveyors 800 tons. The weight of steel in the working house was therefore about 7 lbs. per bushel of storage capacity, while the weight of the circular bins was 2 lbs. per bushel. The total cost, including machinery and foundation was $205,000, of which the working house cost approximately $125,000, and the circular bins $80,000.

Cost of Steel Elevator, Winona Malting Co., Winona, Minn.—The steel working house has a storage capacity of 55,000 bushels, and the 12 steel storage bins are 20 ft. in diameter by 80 ft. high and contain 295,000 bushels. The working house cost about $1.00 per bushel of storage, and the steel bins cost about 14 cents per bushel of storage capacity. The cost of the working house was high on account of the small amount of storage capacity. For a description see Figs. 146 and 147, and Chapter XV.

Miscellaneous Costs.—Eight steel storage tanks, with a capacity of 335,000 bushels in Minnesota cost, including machinery and conveyor galleries, 13 cents per bushel of storage capacity. The Electric Steel storage bins, Minneapolis, Minn., cost, including foundations, about 10 cents per bushel of storage capacity.

Steel working houses will at present (1906) cost from 50 to 80 cents per bushel of storage capacity for large elevators. Steel working houses for small elevators may cost $1.00 to $1.25 per bushel of storage capacity.

A steel grain elevator of the working house and independent storage bin type, with a capacity of 200,000 bushels in the working house and 800,000 bushels in the 8 steel storage bins, has recently been constructed in the middle west for the following unit costs: The steel in the bins and the steel conveyor supports above and below the bins cost 7 cts. per bushel of storage capacity; the conveyor machinery cost 1 ct. per bushel of storage capacity; and the foundations cost 1½ cts. per bushel of storage capacity; making a total cost of 9½ cts. per bushel of storage capacity. The steel in the working house, including leg casings, hoppers, machinery supports, and all steel except the machinery cost 36 cts. per bushel of storage capacity; the machinery, including cleaning machines, motors, etc., cost 15 cts. per bushel of storage capacity; and the foundations cost 3 cts. per bushel of storage capacity; making a total cost of 54 cts. per bushel of storage capacity. These costs are a fair average.

The total cost of the machinery for a 100,000- or a 300,000-bushel steel working house will be approximately the same as for the 200,000-

bushel working house, while the cost of the steel work and foundations will be approximately the same per bushel as for the 200,000-bushel working house. The cost per bushel of the 100,000-bushel working house will be approximately $36 + 30 + 3 = 69$ cts.; while the cost per bushel of the 300,000-bushel working house will be approximately $36 + 7\frac{1}{2} + 3 = 46\frac{1}{2}$ cts.

Steel storage bins cost from 10 to 15 cents per bushel for large bins to 15 to 20 cents per bushel for small bins, varying with local conditions.

A 30,000 bushel storage for a country mill, consisting of four steel tanks, 17½ ft. diameter and 30 feet high, with an interspace bin and a conveyor shed, contained 41 tons of plates and structural steel and 3 tons of corrugated steel, or about 3 lbs. per bushel of grain storage. The shop labor cost, including distribution, was about $15.00 per ton, while the erection, including riveting and painting, cost $19.00 per ton.

Cost of Country Elevators.—Country elevators in the middle west, having a capacity of from 20,000 to 40,000 bushels, will cost complete from 20 to 25 cents per bushel of capacity if built of timber, and 30 to 40 cents per bushel if built of steel.

COST OF REINFORCED CONCRETE GRAIN BINS.—The cost of forms for concrete bins is high per M of lumber, but since each section is raised and used for a complete bin, the cost for forms is not excessively large. Framing the forms costs from $10.00 to $15.00 per M. The concrete in place in the walls will cost from $12.00 to $20.00 per cubic yard. To this must be added the cost of the reinforcing steel.

The timber working house of the Santa Fe Elevator, Chicago, Ill., cost $1.00 per bushel of storage capacity exclusive of power plant, while the reinforced concrete storage bins cost 20 cts. per bushel of storage capacity. The machinery in the working house was unusually expensive, which makes the cost per bushel high. In this connection it should be remembered that the machinery and not the storage capacity is the governing consideration in estimating the cost of a working house.

The working house of the Santa Fe reinforced concrete elevator at Argentine, Kas., cost 60 cts. per bushel of storage capacity, exclusive of power plant; while the reinforced concrete bins cost 20 cts. per bushel of storage capacity.

Cost of Reinforced Concrete Stand Pipe.—The 50′ × 100′ reinforced concrete stand pipe at Attelborough, Mass., cost $34,000 complete, including foundation and roof. This makes 18 cts. per bushel of storage capacity. The walls are 18 inches thick at the bottom, and 8 inches thick at the top. The concrete was 1–2–4 Portland cement concrete and cost in place in the wall per cubic yard as follows:

Cement	$ 4.80
Sand and stone	3.90
Mixing concrete	.40
Placing concrete	2.20
Forms	2.65
Total cost per cu. yd.	$13.95

Bending and placing the steel cost $9.00 per ton, or $2.30 per cubic yard of concrete. There were 770 cubic yards of concrete and 185 tons of steel in the walls.

The cost of moderately large reinforced concrete grain bins is about the same in the middle west as for tile and steel bins. The advantage, if any, in most cases being in favor of the steel bins.

COST OF TILE BINS.—Tile costs from $5.00 to $6.00 per ton at the factory, while the cost of laying the tile will be from 5 to 8 cents per tile. To the cost of the tile must be added the cost of the reinforcing steel and the cement mortar.

In building the tile grain bins 21 feet diameter, for the Canadian Northern Ry., Port Arthur, Ontario, each one thousand tiles took ¾ cubic yard of mortar, and each bricklayer laid 364 tiles per day of 10 hours in the month of November. The entire cost of these bins was 19 cents per bushel of storage capacity. For a description of this elevator and quantities see Chapter XIX.

Tile grain bins cost approximately the same for moderately large bins, under average conditions, as concrete and steel bins, although the steel has the advantage where freights are any considerable item.

APPENDIX I.

DEFINITIONS OF MASONRY TERMS APPLIED TO RAILROAD CONSTRUCTION.*

Masonry.—All constructions of stone or kindred substitute materials in which the separate pieces are either placed together, with or without cementing material to join them, or where not separately placed, are encased in a matrix of firmly cementing material.

Riprapping and paving are not masonry construction.

CLASSIFICATION OF MASONRY.

Kinds of Masonry.	Sub-division into Classes of Material.	Manner of Work.	Dressing.	Face or Surface Finish.
Stone.	Dimension.	In courses.	Fine-pointed Crandalled. Axed or Pean-hammered. Tooth-axed. Sawed. Rubbed.	Quarry-faced. Pitch-faced. Drafted.
	Arch.	In courses.	Fine-pointed. Crandalled. Axed or Pean-hammered. Tooth-axed. Sawed.	
	Ashlar.	Range. Broken Range.		
	Squared stone.	Range Broken range. Random.	Rough-pointed. Fine-pointed. Crandalled.	
	Rubble.	Coursed. Uncoursed.	None.	Quarry-faced.
	Dry.	No mortar.	May be any of the above.	May be any of the above.
Concrete.	Broken stone. Gravel. Other material.			
Brick.	Face. No. 1 paving. No. 2 " Common.	English bond. Flemish bond.		

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* Recommended by American Railway Engineering and Maintenance of Way Association.

Dimension Stone.—A block of stone cut to specified dimensions.

Coping.—A top course of stone or concrete, generally slightly projecting, to shelter the masonry from the weather, or to distribute the pressure from exterior loading.

Arch Masonry.—That portion of the masonry in the arch ring only, or between the intrados and the extrados.

Ashlar Masonry.—Masonry built of ashlar blocks.

Range Masonry.—Masonry in which the various courses are laid up with continuous horizontal beds.

Broken Range Masonry.—Masonry in which the bed joints are parallel but not continuous.

Square-Stone Masonry.—Masonry in which the stones are roughly squared and roughly dressed on beds and sides.

Rubble Masonry.—Masonry composed of squared or roughly squared stones, or rubble of irregular size or shape.

Dry Wall.—A masonry wall in which stones are built up without the use of mortar.

CONCRETE.

Concrete.—A compact mass of broken stone, gravel or other suitable material assembled together with cement mortar and allowed to harden.

Rubble Concrete.—Concrete in which rubblestone are imbedded.

Reinforced Concrete.—Concrete which has been reinforced by means of metal in some form, so as to give the concrete elasticity and increased strength.

BRICK.

Brick.—No. 1.—Hard burned brick, absorption not to exceed 2 per cent by weight.

Brick.—No. 2.—Softer and lighter brick than No. 1, absorption not to exceed 6 per cent by weight.

CEMENT.

Cement.—A preparation of calcined clay and limestone, or their equivalents, possessing the property of hardening into a solid mass when moistened with water. This property is exercised under water, as well as in the air. Cements are divided into four classes: Portland, Natural, Puzzolan and Silica cement. (See each.)

Portland Cement.—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent has been made subsequent to calcination.

Natural Cement.—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

Puzzolan.—An intimate mixture of pulverized granulated furnace slag and slaked lime without further calcination which possesses the hydraulic qualities of cement.

Silica Cement (*sand cement*).—A mixture of clean sand and Portland cement ground together.

Descriptive Words.

Arris.—An external angle, edge or ridge.

Ashlar.—A squared or cut block of stone with rectangular dimensions.

Axed.—Dressed so as to cover the surface of a stone with chisel marks which are nearly or quite parallel.

Backing.—That portion of a masonry wall or structure built in the rear of the face. It must be attached to the face and bonded with it. It is usually of a cheaper grade of masonry than the face.

Batter.—The slope or inclination of the face from a vertical line.

Bed.—The top and bottom of a stone. (See Course Bed; Natural Bed; Foundation Bed.)

Bed Joint.—A horizontal joint, or one perpendicular to the line of pressure.

Beton.—(See Concrete.)

Block Rubble.—Large blocks of building stone as they come from the quarry. (See Rubble.)

Bond.—The mechanical disposition of stone, brick or other building blocks by overlapping to break joints.

Build.—A vertical joint.

Bush-Hammered.—A surface produced by removing the roughness of stone with a bush-hammer.

Centering.—A temporary support used in arch construction. (Also called Centers.)

Course.—Each separate layer in stone, concrete or brick masonry.

Course Bed.—Stone, brick or other building material in position, upon which other material is to be laid.

Crandalled.—Dressed with a crandalling tool, producing the same effect as fine-pointed.

Draft.—A line on the surface of a stone cut to the breadth of the chisel.

Draft Stones.—Stones on which the face is surrounded by a draft, the space inside the draft being left rough.

Dressing.—The finish given to the surface of stones or to concrete.

Expansion Joint.—A vertical joint or space to allow for temperature changes.

Extrados.—The upper or convex surface of an arch.

Face.—The exposed surface in elevation.

Facing.—In concrete: (1) A rich mortar placed on the exposed surfaces to make a smooth finish. (2) Shovel facing by working the mortar of concrete to the face.

Final Set.—A stage of the process of setting marked by certain hardness. (See Cement Specifications.)

Fine Pointed.—Dressed by fine point to smoother finish than by rough point.

Flush.—(Adj.) Having the surface even or level with an adjacent surface. (Verb.) (1) To fill. (2) To bring to a level. (3) To force water to the surface of mortar or concrete by compacting or ramming.

Footing.—A projecting bottom course.

Forms.—Temporary structures for holding concrete in desired shape.

Foundation.—(1) That portion of a structure, usually below the surface of the ground, which distributes the pressure upon its support. (2) Also applied to the natural support itself; rock, clay, etc.

Foundation Bed.—The surface on which a structure rests.

Grout.—A thin mortar either poured or applied with a brush.

Header.—A stone which has its greatest length at right angles to the face of the wall, and which bonds the face stones to the backing.

Initial Set.—An early stage of the process of setting, marked by certain hardness. (See Cement Specifications.)

Intrados.—The inner or concave surface of an arch.

Joint.—The narrow space between adjacent stones, bricks or other building blocks, usually filled with mortar.

Lagging.—Strips used to carry and distribute the weight of an arch to the ribs or centering during its construction.

Leveller.—A small rectangular stone, not less than four to six inches thick, used in broken range work to complete the bed for a stone in the course above and give it proper bond. Sometimes called jumper or dutchman.

Lock.—Any special device or method of construction used to secure a bond in work.

Mortar.—A mixture of sand, cement or lime and water, used to cement the various stones or brick in masonry or to cover the surface of same.

Natural Bed.—The surface of a stone parallel to its stratification.

Paving.—Regularly placed stone or brick forming a floor.

Pean-Hammered.—(See Axed.)

Pinner.—A spall or small stone used to wedge up a stone and give it better bearing.

Pitched-Faced.—Having the arris clearly defined by a line beyond which the rock is cut away by the pitching chisel so as to make approximately true edges.

Pointing.—Filling joints or defects in the face of a masonry structure.

Quarry-Faced.—Stone faced as it comes from the quarry.

Random Range Masonry.—(See Broken Range Masonry.)

Riprap.—Rough stone of various sizes placed compactly or irregularly to prevent scour by water.

Rough Pointed.—Dressed by pick or heavy point until the projections vary from one-half to one inch.

Rubbed.—A fine finish made by rubbing with grit or sandstone.

Rubble.—Field stone or rough stone as it comes from the quarry. When it is of large or massive size it is termed block rubble.

Set (noun).—The change from a plastic to a solid or hard state.

Soffit.—The under side of a projection.

Spall (noun).—A chip or small piece of stone broken from a large block.

Stretcher.—A stone which has its greatest length parallel to the face of the wall.

Tooth-Axed.—Dressed by a method of fine pointing.

Voussoirs.—The stones, blocks of concrete or other material forming the arch ring.

SPECIFICATIONS FOR STONE MASONRY.*

GENERAL.

1. *Stone.*—Stone masonry shall be built of the kinds specially designated, with such arrangements of courses and bond as shown on the drawings or as directed. The stone shall be hard and durable, free from seams or other imperfections, of approved quality and shape, and in no case have less bed than rise, and shall be laid on their broadest beds, well bonded and solidly bedded. When liable to be affected by freezing, no unseasoned stone shall be laid.

2. *Dressing.*—Dressing must be the best of the kind specified for each class of work.

3. *Beds.*—Beds and joints or builds must be square with each other, and dressed true and out of wind. Hollow beds will not be allowed.

4. *Dressing.*—All stone must be dressed for laying on natural beds.

5. *Drafts.*—Margin drafts must be neat and accurate.

6. *Pitching.*—Pitching must be done to true lines and exact batter.

7. *Mixing.*—The sand and cement should be mixed dry and in small batches in proportions as directed, on a suitable platform, which must be kept clean and free from all foreign matter; then water is to be added, and the whole mixed until the mass of mortar is thoroughly homogeneous and leaves the hoe clean when drawn from it. It must not be retempered after it has begun to set.

8. *Laying.*—All stones must be laid on natural beds. Each stone must be settled into place in full bed of mortar.

9. No stone must be dropped or slid over the wall, but must be placed without jarring the stones already laid.

10. No heavy hammering will be allowed on the wall after a course is laid.

11. If a stone becomes loose after the mortar is set, it must be relaid with fresh mortar.

12. Each stone must be cleansed and dampened before laying.

13. *Laying in Freezing Weather.*—Stones must not be laid in freezing weather unless allowed by the engineer. If allowed, they must be freed from ice, snow or frost by warming, and laid in mortar made of heated sand and water, or, with proper precautions, mixed with brine in proportions of one pound of salt to eighteen gallons of water, when the temperature is thirty-two degrees Fahrenheit. Add one

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* Adopted by the American Railway Engineering and Maintenance of Way Association in 1906.

ounce of salt for every degree of temperature below thirty-two degrees Fahrenheit.

14. Stones must be laid to exact lines and levels so as to give the required bond and thickness of mortar in beds and joints.

15. *Pointing.*—Mortar in beds and joints of exposed faces must be removed to a depth of one inch before it has set. No pointing shall be done until the wall is complete and mortar set, nor when frost is in the stone. Wet the joints and fill again with mortar made of equal parts sand and Portland cement. It must be pounded in with "set-in" or calking tool, and finished with a beading tool the width of the joint, used with a straight edge.

CLASSIFICATION.

16. Stone masonry will be classified under the following heads: Bridge and Retaining Wall Masonry, Arch Masonry, Culvert Masonry and Dry Masonry.

BRIDGE AND RETAINING WALL MASONRY.

17. *Classes.*—Bridge and retaining wall masonry shall consist of two classes: (a) Ashlar (either coursed or broken coursed) and (b) rubble.

(*a*) ASHLAR BRIDGE AND RETAINING WALL MASONRY.

18. *Stone.*—In ashlar bridge and retaining wall masonry (either coursed or broken coursed), the stone must be large and well proportioned.

19. *Courses.*—No course to be less than 14 inches nor more than 30 inches thick, the thickness of courses to diminish regularly from bottom to top.

20. *Dressing.*—The beds and joints or builds of face stones shall be fine pointed, so that the mortar layer shall not exceed one-half an inch in thickness when the stones are laid.

21. *Joints.*—Joints in face stones must be full to the square for a depth equal to at least one-half the height of the course, but in no case less than 12 inches.

22. *Surface Finish.*—The exposed surface of each face stone will be rock faced, and the edges pitched to true lines and exact batter; the face to have no projections over 3 inches beyond the pitch lines.

23. *Chisel Draft.*—A chisel draft one and one-half inches wide shall be cut at each exterior corner.

24. *Handling.*—No holes for stone hooks will be permitted to show in exposed surfaces. They must be handled with clamps, keys, lewis or dowels.

25. *Stretchers.*—Stretchers shall be not less than 4 feet long, and to have at least one and one-fourth times as much bed as thickness of the course.

26. *Headers.*—Headers shall be not less than 4 feet in length. They shall occupy one-fifth of the face of the wall, and no header shall have less than 18 inches width of face, and where the course exceeds 18 inches in height the width of face shall not be less than the height of course. Headers shall hold the size in the heart of the wall that they show on the face, and be so arranged that a header in a superior course shall be placed between two headers in a course below; but no header shall be laid over a joint, and no joint shall occur over a header. They shall be similarly disposed in the back of the wall, interlocking with those in the face when the thickness of the wall will admit. When the wall is too thick to admit of such arrangement, stones of not less than 4 feet in length shall be placed transversely in the heart of the wall to bond the two opposite sides of it.

27. *Backing.*—Backing shall be large, well shaped stone, roughly bedded and jointed, the bed joints not to exceed 1 inch and vertical joints generally not to exceed 2 inches. No part or portion of vertical joints shall have a greater dimension than 6 inches, which void shall be thoroughly filled with spalls full bedded in cement mortar or filled with concrete. At least one-half of the backing stones shall be of the same size and character as the face stone and with parallel beds.

28. When face stone is backed with two courses, neither course shall be less than 8 inches thick.

29. When the wall is 3 feet thick or less, the face stone shall pass entirely through and no backing be allowed.

30. If the engineer so directs, the backing may be entirely of concrete, or back laid with headers and stretchers, as specified above, and heart of wall filled with concrete.

31. The bond of stone on face, back and heart of wall shall not be less than 12 inches. Backing shall be laid to break joints with the face stone and with one another.

32. *Coping.*—Coping will be dimension stone, holding full size throughout, proportioned for its loading, as marked on the drawings.

33. The beds, joints and top will be fine pointed.

34. The location of joints will be determined by the position of the bed plates, and must be shown on the drawings.

35. When required, in the judgment of the engineer, coping stones, stones in the wings of abutments and the stones on piers shall be secured together with iron cramps or dowels, their position being indicated by the engineer.

(*b*) Rubble Bridge and Retaining Wall Masonry.

36. *Stones.*—Rubble bridge and retaining wall masonry will consist of stones roughly squared, laid in irregular courses. The beds must be parallel, roughly dressed, and lie horizontally in the wall. The bottom stones shall be large, selected flat stones. The wall must be compactly laid, having at least one-fifth the surface of the back and face headers, so arranged as to interlock, having all the spaces in the heart of the wall filled with suitable stones and spalls, thoroughly bedded in cement mortar or filled with concrete. The face joints must not be more than 1 inch in thickness.

Arch Masonry.

37. Arch masonry shall consist of the arch ring only, or that portion between the intrados and extrados, and shall be of two classes: (a) Ashlar Arch Masonry, and (b) Rubble Arch Masonry.

(*a*) Ashlar Arch Masonry.

38. *Voussoirs.*—The voussoirs must be full size throughout, and must have bond not less than thickness of the stone and dressed true to templet.

39. The number of courses and depth of voussoirs will be shown on the drawings.

40. *Joints.*—The joints of the voussoirs and the intrados shall be fine pointed. Mortar joints shall not exceed three-eighths of an inch.

41. *Finish.*—The exposed surface of the ring stone shall be smooth or rock faced, with a marginal draft.

42. Voussoirs shall be carried up simultaneously from both bench walls.

43. *Backing.*—Backing may consist of large stones shaped to fit the arch bonded to the spandrel and laid in full beds of mortar. Concrete may also be used for backing.

44. *Waterproofing.*—If waterproofing is required, a thin coat of

mortar or grout shall be applied for a finishing coat, upon which shall be placed a covering of suitable waterproofing material.

45. *Centers.*—Centers shall not be struck until directed.

46. *Miscellaneous Walls.*—Bench walls, piers, spandrels, parapets, with walls and copings will be built under the specifications for ashlar bridge and retaining wall masonry.

(*b*) RUBBLE ARCH MASONRY.

47. *Voussoirs.*—The voussoirs must be full size throughout, and must have bond not less than thickness of the stone.

48. The depth of voussoirs will be shown on the drawings.

49. The beds need only be roughly dressed so as to bring them to radial planes.

50. *Joints.*—Mortar joints shall not exceed 1 inch.

51. *Finish.*—The exposed surface of the ring stones shall be rock faced and the edges pitched to true lines.

52. Voussoirs shall be carried up simultaneously from both bench walls.

53. *Backing.*—Backing may consist of large stones shaped to fit the arch bonded to the spandrel and laid in full beds of mortar. Concrete may also be used for backing.

54. *Waterproofing.*—If waterproofing is required, a thin coat of mortar or grout shall be applied for a finishing coat, upon which shall be placed a covering of suitable waterproofing material.

55. *Centers.*—Centers shall not be struck until directed.

56. *Miscellaneous Walls.*—Bench walls, piers, spandrels, parapets and wing walls will be built under the specifications for rubble bridge and retaining wall masonry.

CULVERT MASONRY.

57. *Stone.*—Culvert masonry must be laid in cement mortar. The character of stone used and quality of work must be similar to that specified for rubble bridge and retaining wall masonry.

58. *Headers.*—One-half the top stones of the side walls must extend entirely across the wall.

59. *Cover Stones.*—The covering must be of sound, strong stone, at least 12 inches thick, or as shown on drawings. They must be roughly dressed so as to make close joints with each other, and must lap their whole width at least 12 inches over the side walls. They must be

doubled under high embankments, as directed by the engineer or shown on the drawings.

60. *End Walls.*—The end walls must be covered with suitable coping.

61. *Dry Masonry.*—Dry masonry will include dry retaining walls and slope walls.

Dry Retaining Walls.

62. Dry retaining walls will include all dry rubble work for retaining embankments or similar work.

63. *Stone.*—Flat stone at least twice as wide as thick will be used. Beds and joints to be roughly dressed square to each other and to face of stone.

64. *Joints.*—Joints not to exceed three-fourths of an inch.

65. *Sizes of Stone.*—The different sizes of stone must be evenly distributed over the whole face of wall, generally keeping the largest stone in the lower part of the wall.

66. *Laying.*—The work shall be well bonded and present a reasonably true and smooth surface, free from holes or projections. This wall is double faced and self-sustaining.

Slope Walls.

67. *Stone.*—Slope walls shall be built of such thickness and slope as may be required by the engineer. No stone shall be used in their construction which does not reach through the wall. Stones to be placed at right angles to the slopes. This wall is single faced and built with steep slopes simultaneously with the embankment which it is to protect.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT CONCRETE.*

1. *Cement.*—Cement shall be Portland, either American or foreign, which will meet the requirements of the standard specifications.

2. *Sand.*—Sand shall be clean, sharp, coarse, and of grains varying in size. It shall be free from sticks and other foreign matter, but it may contain clay or loam not to exceed five (5) per cent. Crusher dust, screened to reject all particles over one-quarter (¼) in. in diameter, may be used instead of sand if approved by the engineer.

3. *Stone.*—Stone shall be sound, hard and durable, crushed to sizes not exceeding two inches in any direction. For reinforced concrete sizes usually are not to exceed three-quarters (¾) in. in any direction, but may be varied to suit character of reinforcing material.

4. *Gravel.*—Gravel shall be composed of clean pebbles of hard and durable stone of sizes not exceeding two inches in diameter, and shall be free from clay and other impurities except sand. When containing sand in any considerable quantity, the amount of sand per unit of volume of gravel shall be determined accurately, to admit of the proper proportion of sand being maintained in the concrete mixture.

5. *Water.*—Water shall be clean and reasonably clear, free from sulphuric acid or strong alkalies.

6. *Mixing by Hand.*—(a) Tight platforms shall be provided of sufficient size to accommodate men and materials for the progressive and rapid mixing of at least two batches of concrete at the same time. Batches shall not exceed one cubic yard each, and smaller batches are preferable, based upon a multiple of the number of sacks of cement to the barrel.

(*b*) Spread the sand evenly upon the platform, then the cement upon the sand and mix thoroughly until of an even color. Add all the water necessary to make a thin mortar and spread again; add the gravel if used, and finally the broken stone, both of which, if dry, should first be thoroughly wet down. Turn the mass with shovels or hoes until thoroughly incorporated, and all the gravel and stone is covered with mortar; this will probably require the mass to be turned four times.

(*c*) Another approved method, which may be permitted at the option of the engineer in charge, is to spread the sand, then the cement and mix dry, then the gravel or broken stone; add water and mix thoroughly as above.

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* Adopted by the American Railway Engineering and Maintenance of Way Association, 1904.

7. *Mixing by Machine.*—A machine mixer shall be used wherever the volume of work will justify the expense of installing the plant. The necessary requirements for the machine will be that a precise and regular proportioning of materials can be controlled and that the product delivered shall be of the required consistency and thoroughly mixed.

8. *Consistency.*—The concrete shall be of such consistency that when dumped in place it will not require much tamping. It shall be spaded down and tamped sufficiently to level off, and the water should rise freely to the surface.

9. *Forms.*—(*a*) Forms shall be well built, substantial and unyielding, properly braced or tied together by means of wire or rods, and shall conform to lines given.

(*b*) For all important work, the lumber used for face work shall be dressed on one side and both edges to a uniform thickness and width, and shall be sound and free from loose knots, secured to the studding or uprights in horizontal lines.

(*c*) For backings and other rough work undressed lumber may be used.

(*d*) Where corners of the masonry and other projections liable to injury occur, suitable moldings shall be placed in the angles of the forms to round or bevel them off.

(*e*) Lumber once used in forms shall be cleaned before being used again.

(*f*) The forms must not be removed within thirty-six hours after all the concrete in that section has been placed. In freezing weather they must remain until the concrete has had a sufficient time to become thoroughly set.

(*g*) In dry but not freezing weather, the forms shall be drenched with water before the concrete is placed against them.

10. *Disposition.*—(*a*) Each layer should be left somewhat rough to insure bonding with the next layer above; and, if it be already set, shall be thoroughly cleaned and scrubbed with coarse brushes and water before the next layer is placed upon it.

(*b*) Concrete shall be deposited in the molds in layers of such thickness and position as shall be specified by the engineer in charge. Temporary planking shall be placed at ends of partial layers, so that none shall run out to a thin edge. In general, excepting in arch work, all concrete must be deposited in horizontal layers throughout.

(*c*) The work shall be carried up in sections of convenient length and each section completed without intermission.

(*d*) In no case shall work on a section stop within 18 inches of the top.

(*e*) Concrete shall be placed immediately after mixing, and any having an initial set shall be rejected.

11. *Expansion Joints.*—(a) In exposed work, expansion joints may be provided at intervals of 30 to 100 ft., as the character of the structure may require.

(*b*) A temporary vertical form or partition of plank shall be set up and the section behind completed as though it were the end of the structure. The partition shall be removed when the next section is begun, and the new concrete placed against the old without mortar flushing. Locks shall be provided if directed or called for by the plans.

(*c*) In reinforced concrete the length of these sections may be materially increased at the option of the engineer.

12. *Facing.*—(*a*) The facing will be made by carefully working the coarse material back from the form by means of a shovel, bar or similar tool, so as to bring the excess mortar of the concrete to the face.

(*b*) About one inch of mortar (not grout) of the same proportions as used in the concrete may be placed next to the forms immediately in advance of the concrete.

(*c*) Care must be taken to remove from the inside of the forms any dry mortar in order to secure a perfect face.

13. *Proportioning.*—The proportion of the materials in the concrete shall be as specifically called for by the contract, or as set forth herein, upon the lines left for that purpose; the volume of cement to be based upon the actual cubic contents of one barrel of specified weight.

Structure.	**Parts by Volume.**			
	Cement.	**Sand.**	**Gravel.**	**Broken Stone.**

14. *Finishing.*—(*a*) After the forms are removed, which should generally be as soon as possible after the concrete is sufficiently set, any small cavities or openings in the face shall be neatly filled with mortar, if necessary in the opinion of the engineer. Any ridges due to cracks or points in the lumber may be rubbed down with a chisel or wooden float. The entire face may then be washed with a thin grout of the consistency of whitewash, mixed in the same proportion as the mortar of the concrete. The wash should be applied with a brush. The earlier the above operations are performed the better will be the result.

(*b*) The tops of bridge seats, pedestals, copings, wing walls, etc., when not finished with natural stone coping, shall be finished with a smooth surface composed of one part cement to two parts of granite, or other suitable screenings or sand, applied in a layer one-half ($\frac{1}{2}$) to one (1) in. thick. This must be put in place with the last course of concrete.

15. *Waterproofing.*—Where waterproofing is required, a thin coat of mortar or grout shall be applied for a finishing coat, upon which shall be placed a covering of suitable waterproofing material.

16. *Freezing Weather.*—Ordinarily concrete to be left above the surface of the ground will not be constructed in freezing weather. Portland cement concrete, however, may be built under these conditions by special instructions. In this case the sand, water and broken stone shall be heated, and in severe cold salt shall be added in the proportion of about two pounds per cubic yard.

17. *Reinforced Concrete.*—Where concrete is deposited in connection with metal reinforcing, the greatest care must be taken to insure the coating of the metal with cement, and the thorough compacting of the concrete around the metal. Wherever it is practicable the metal should be placed in position first. This can usually be done in the case where the metal occurs in the bottoms of the forms by supporting the same on transverse wires, or otherwise, when the bottoms of the forms can be flushed with cement mortar, so as to get the mortar under the metal at the same time, and the concrete deposited immediately afterward. The mortar for flushing the bars should be composed of one part cement and two parts sand. The metal used in the concrete shall be free from dirt, oil or grease. All mill scale should be removed by hammering the metal, or preferably by pickling the same in a weak solution of muriatic acid. No salt will be used in reinforced concrete when laid in freezing weather.

APPENDIX II.

SPECIFICATIONS FOR MATERIAL AND WORKMANSHIP FOR STEEL STRUCTURES.*

MATERIAL.

1. **Process of Manufacture.**—Steel shall be made by the open-hearth process.

2. **Schedule of Requirements.**

Chemical and Physical Properties.	Structural Steel.	Rivet Steel.	Steel Castings.
Phosphorus Max.. Basic ...	0.04 per cent.	0.04 per cent.	0.05 per cent.
Phosphorus Max.. Acid.....	0.08 " "	0.04 " "	0.08 " "
Sulphur maximum............	0.05 " "	0.04 " "	0.05 " "
Ultimate tensile strength.. Pounds per square inch......	Desired 60000	Desired 50000	Not less than 65000
Elongation: min. % in 8″	$\frac{1,500,000}{\text{Ult. tensile strength}}$ †	$\frac{1,500,000}{\text{Ult. tensile strength.}}$	
Elongation: min. % in 2″...	22		18
Character of fracture.........	Silky	Silky	Silky or fine granular
Cold bends without fracture.	180° flat ‡	180° flat §	90°, $d = 3t$

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

3. **Allowable Variations.**—If the ultimate strength varies more than 4,000 lbs. from that desired, a retest shall be made on the same gage, which, to be acceptable, shall be within 5,000 lbs. of the desired ultimate.

4. **Chemical Analyses.**—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent above the required limits will be allowed.

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* Adopted by American Railway and Maintenance of Way Association, and the American Society for Testing Materials.

† See paragraph 11. ‡ See paragraphs 12, 13 and 14. § See paragraph 15.

5. **Form of Specimens.** PLATES, SHAPES AND BARS.—Specimens for tensile and bending tests for plates, shapes and bars shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ inch for a length of at least 9 inches, with enlarged ends.

6. RIVETS.—Rivet rods shall be tested as rolled.

7. PINS AND ROLLERS.—Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be 1 inch from the surface of the bar. The specimen for tensile test shall be turned to the form shown by Fig. 2. The specimen for bending test shall be 1 inch by ½ inch in section.

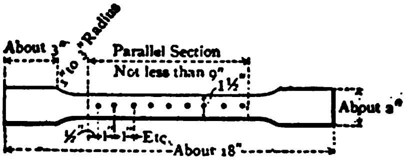

FIG. 1

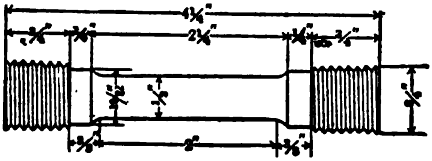

FIG. 2.

8. STEEL CASTINGS.—The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons molded and cast on some portion of one or more castings from each melt or from the sink heads, if the heads are of sufficient size. The coupon or sink head, so used, shall be annealed with the casting before it is cut off. Test specimens to be of the form prescribed for pins and rollers.

9. **Annealed Specimens.**—Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimens for tensile tests representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

10. **Number of Tests.**—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ inch and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

11. **Modifications in Elongation.**—For material less than 5⁄16 inch and more than ¾ inch in thickness the following modifications will be allowed in the requirements for elongation:

(*a*) For each 1⁄16 inch in thickness below 5⁄16 inch, a deduction of 2½ per cent will be allowed from the specified elongation.

(*b*) For each ⅛ inch in thickness above ¾ inch, a deduction of 1 per cent will be allowed from the specified elongation.

(*c*) For pins and rollers over 3 inches in diameter the elongation in 8 inches may be 5 per cent less than that specified in paragraph 2.

12. **Bending Tests.**—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than 1 inch thick shall bend as called for in paragraph 2.

13. **Thick Material.**—Full-sized material for eye-bars and other steel 1 inch thick and over, tested as rolled, shall bend cold 180 degrees around a pin the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend.

14. **Bending Angles.**—Angles ¾ inch and less in thickness shall open flat and angles ½ inch and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

15. **Nicked Bends.**—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky, uniform fracture.

16. **Finish.**—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and have a smooth, uniform, workmanlike finish. Plates 36 inches in width and under shall have rolled edges.

17. **Stamping.**—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached metal tag.

18. **Defective Material.**—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious

defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

19. **Allowable Variation in Weight.**—A variation in cross-section or weight of each piece of steel of more than 2½ per cent from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates.

20. **When Ordered to Weight.**—Plates 12½ pounds per square foot or heavier:

(*a*) Up to 100 inches wide, 2½ per cent above or below the prescribed weight.

(*b*) One hundred inches wide and over, 5 per cent above or below.

21. Plates under 12½ pounds per square foot:

(*a*) Up to 75 inches wide, 2½ per cent above or below.

(*b*) Seventy-five inches and up to 100 inches wide, 5 per cent above or 3 per cent below.

(*c*) One hundred inches wide and over, 10 per cent above or 3 per cent below.

PLATES ¼ INCH AND OVER IN THICKNESS.

Thickness Ordered.	Nominal Weight.	Width of Plate.			
		Up to 75 Inch.	75" and up to 100".	100" and up to 115."	Over 115."
1-4 inch.	10.20 lbs.	10 per cent.	14 per cent.	18 per cent.	
5-16 "	12.75 "	8 " "	12 " "	16 " "	
3-8 "	15.30 "	7 " "	10 " "	13 " "	17 per cent.
7-16 "	17.85 "	6 " "	8 " "	10 " "	13 " "
1-2 "	20.40 "	5 " "	7 " "	9 " "	12 " "
9-16 "	22.95 "	4½ " "	6½ " "	8½ " "	11 " "
5-8 "	25.50 "	4 " "	6 " "	8 " "	10 " "
Over 5-8 "		3½ " "	5 " "	6½ " "	9 " "

PLATES UNDER ¼ INCH IN THICKNESS.*

Thickness Ordered.	Nominal Weights Lbs. per Square Ft.	Width of Plate.		
		Up to 50."	50" and up to 70"	Over 70."
1-8" up to 5-32"	5.10 to 6.37	10 per cent.	15 per cent.	20 per cent.
5-32 " " 3-16	6.37 " 7.65	8½ " "	12½ " "	17 " "
3-16 " " 1-4	7.65 " 10.20	7 " "	10 " "	15 " "

* Withdrawn but was adopted by American Society for Testing Materials, September 1, 1905.

22. **When Ordered to Gage.**—Plates will be accepted if they measure not more than .01 inch below the ordered thickness.

23. An excess over the nominal weight, corresponding to the dimensions on the order, will be allowed for each plate, if not more than that shown in the preceding tables, one cubic inch of rolled steel being assumed to weigh 0.2833 pounds.

SPECIAL METALS.

24. **Cast-Iron.**—Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar, 1¼ in. in diameter and 15 in. long. The transverse test shall be on a supported length of 12 in. with load at middle. The minimum breaking load so applied shall be 2,900 lbs., with a deflection of at least $\frac{1}{10}$ inch before rupture.

25. **Wrought-Iron Bars.**—Wrought-iron shall be double-rolled, tough, fibrous, and uniform in character. It shall be thoroughly welded in rolling and be free from surface defects. When tested in specimens of the form of Fig. 1, or in full-sized pieces of the same length, it shall show an ultimate strength of at least 50,000 lbs. per sq. in., an elongation of at least 18 per cent in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber through 135°, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent the fracture shall show at least 90 per cent fibrous.

INSPECTION AND TESTING AT THE MILLS.

26. The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

27. The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

28. When an inspector is furnished by the purchaser to inspect mate-

rial at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

WORKMANSHIP.

29. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

30. **Straightening Material.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

31. **Finish.**—Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

32. **Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

33. **Rivet Holes.**—When general reaming is not required, the diameter of the punch for material not over $\frac{5}{8}$ inch thick shall be not more than $\frac{1}{16}$ inch, nor that of the die more than $\frac{1}{8}$ inch larger than the diameter of the rivet. The diameter of the die shall not exceed that of the punch by more than $\frac{1}{4}$ the thickness of the metal punched. Material over $\frac{5}{8}$ inch thick, except minor details, and all material where general reaming is required, shall be sub-punched and reamed as per paragraph 62, or drilled from the solid. Holes in flanges of rolled beams and channels used in floors of railroad bridges shall be drilled from the solid. Those in webs of same shall be so drilled or sub-punched and reamed.

34. **Punching.**—Punching shall be accurately done. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection at the option of the inspector.

35. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces to be painted (see paragraph 66).

36. **Lattice Bars.**—Lattice bars shall have neatly rounded ends, unless otherwise called for.

37. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

38. **Splice Plates and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ inch of flange angles.

39. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than $\frac{1}{8}$ inch, unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$ inch clearance between ends of plates will be allowed.

40. **Connection Angles.**—Connection angles for floor girders shall be flush with each other and correct as to position and length of girder. In case milling is required after riveting, the removal of more than $\frac{1}{16}$ inch from their thickness will be cause for rejection.

41. **Riveting.**—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

42. Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets great care shall be taken not to injure the adjacent metal. If necessary they shall be drilled out.

43. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts turned to a driving fit. A washer not less than $\frac{1}{4}$ inch thick shall be used under nut.

44. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

45. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints depending on contact bearing the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

46. **Field Connections.**—Holes for floor girder connections shall be sub-punched and reamed with twist drills to a steel template 1 inch thick. Unless otherwise allowed, all other field connections shall be assembled in the shop and the unfair holes reamed; and when so reamed the pieces shall be match-marked before being taken apart.

47. **Eye-Bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$ inch from the thickness of the bar.

48. **Boring Eye-Bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ inch smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

49. **Pin Holes.**—Pin holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

50. The distance center to center of pin holes shall be correct within $\frac{1}{32}$ inch, and the diameter of the hole not more than $\frac{1}{50}$ inch larger than that of the pin, for pins up to 5 inches diameter, and $\frac{1}{32}$ inch for larger pins.

51. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gages and shall be straight and smooth and entirely free from flaws.

52. **Pilot Nuts.**—At least one pilot and driving nut shall be furnished for each size of pin for each structure, and field rivets 10 per cent in excess of the number of each size actually required.

53. **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of $1\frac{3}{8}$ inch, when they shall be made with six threads per inch.

54. **Annealing.**—Steel, except in minor details, which has been partially heated shall be properly annealed.

55. **Steel Castings.**—All steel castings shall be annealed.

56. **Welds.**—Welds in steel will not be allowed.

57. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

58. **Shipping Details.**—Pins, nuts, bolts, rivets, and other small details shall be boxed or crated.

59. **Weight.**—The weight of every piece and box shall be marked on it in plain figures.

60. **Finished Weight.**—Payment for pound price contracts shall be by scale weight. No allowance over 2 per cent of the total weight of the structure as computed from the plans will be allowed for excess weight.

Additional Specifications When General Reaming and Planing are Required.

61. **Planing Edges.**—Sheared edges and ends shall be planed off at least $\frac{1}{4}$ inch.

62. **Reaming.**—Punched holes shall be made with a punch $\frac{3}{16}$ inch smaller in diameter than the nominal size of the rivets and shall be reamed to a finished diameter of not more than $\frac{1}{16}$ inch larger than the rivet.

63. **Reaming after Assembling.**—Wherever practicable, reaming shall be done after the pieces forming one built member have been assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

64. **Removing Burrs.**—The burrs on all reamed holes shall be removed by a tool countersinking about $\frac{1}{16}$ inch.

Shop Painting.

65. Steel work, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

66. In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

67. Pieces and parts which are not accessible for painting after erection, including tops of stringers, eye-bar heads, ends of posts and chords, etc., shall have a good coat of paint before leaving the shop.

68. Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

69. Machine finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

INSPECTION AND TESTING AT THE SHOPS.

70. The manufacturer shall furnish all facilities for inspecting and testing weight and the quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members if required.

71. The purchaser shall be furnished complete shop plans, and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship. Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

72. When an inspector is furnished by the purchaser, he shall have full access, at all times, to all parts of the shop where material under his inspection is being manufactured.

73. The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

FULL-SIZED TESTS.

74. Full-sized parts of the structure may be tested at the option of the purchaser. Such tests on eye-bars and similar members to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser, at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected. The expense of testing members, to prove their design, shall be paid for by the purchaser.

75. In eye-bar tests the ultimate strength, true elastic limit and the elongation in 10 feet, unless a different length is called for, shall be recorded.

76. In transverse tests the lateral and vertical deflections shall be recorded.

UNIT STRESSES.*

All parts of the structure shall be so proportioned that the sum of the maximum stresses shall not exceed the following amounts in pounds per square inch of cross-section:

Tension	16,000
Compression	$16{,}000 - 70\frac{l}{r}$

where l = length of the member in inches and r = the least radius of gyration in inches.

Rivets and pins, bearing	22,000
Rivets and pins, shear	11,000
Pins, bending on extreme fiber	24,000
Plate girder webs, shearing on net section	10,000
Bearing on granite masonry	500
Bearing on sandstone, limestone, and Portland cement concrete masonry	300
Expansion rollers per lineal inch	$600D$

where D = diameter of roller.

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* Recommended by the American Railway Engineering and Maintenance of Way Association, but not formally adopted.

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